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**PRACTICAL
WATER-POWER ENGINEERING**

PRACTICAL WATER-POWER ENGINEERING

A TREATMENT OF TECHNICAL AND COMMERCIAL FACTORS; WITH SPECIAL REFERENCE TO FIELD WORK INVOLVING STUDY OF STREAM FLOW, PONDAGE CAPACITY, DEVELOPED WATERWAYS, POWER TRANSMISSION LINES, ETC., ETC.

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PREFACE

IN writing this book much information has been drawn upon which appeared in a series of articles published in the *Electrical Review* and an article in *Water Engineering* during the year 1923. The aim has been rather to treat with water-power engineering from experience in the field, and to bring forward matter of prime importance and to point out the most particular conditions to be met with in water-power development. Designs of waterways, such as gravity conduits and supply pipe lines, are discussed, and charts given to facilitate calculation for sizes. Turbine designs, which really start and end at the factory, are not treated. The subject treats at some length with conditions affecting the study and choice of sites, the location of hydraulic structures, and the economics of electric power transmission.

Chapter II. takes up the study of rainfall and run-off, and presents methods for determining the probable run-off for different drainage areas and characteristics. Chapter III. brings out the many diverse points on the selection of water-power sites. Chapter IV. deals with storage and forebay capacity. Chapter V. treats at some length with stream-flow conditions and the determination of flow for channels of different hydraulic properties and characteristics. Chapter VI. deals with the gravity conduit, and gives a chart for computing the mean velocity of flow into terms of hydraulic radius, coefficient of roughness, and slope of conduit bottom. Chapter VII. takes up the question of riveted steel supply pipe lines, and presents charts for determining their size, thickness, velocity, and flow, etc. Chapter VIII. deals with the hydraulic losses, commencing with the catchment or drainage area. Chapter IX. helps to point out the advantages of developing automatically operated small water powers, which are now going

to waste. In the study of water powers, the investigating engineer is liable to overlook certain important points bearing on the particular project, and Chapter X., as also parts of the above-mentioned chapters, indicate many points likely to be involved.

Chapter XII. takes up the question of very large overhead power transmission lines, and emphasises the advantages of earthing the neutral of power transformers and also working out the design of lines based on the economic line-voltage and spans of line-supports. Chapter XIII. treats with the usual design of overhead power lines, and presents formulæ and a chart for determining the spacing and size of line-conductors, etc. Chapter XIV. shows the various factors involved in overhead power transmission line economics.

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CONTENTS

CHAPTER	PAGE
I. HYDRAULICS AND WATER-POWER DEVELOPMENT - - -	1
II. CATCHMENT, RAINFALL, AND RUN-OFF - - -	16
III. SELECTION OF WATER-POWER SITES - - -	31
IV. STORAGE AND PONDAGE - - -	47
V. STREAM FLOW - - -	70
VI. THE GRAVITY CONDUIT - - -	93
VII. PRESSURE PIPE LINES - - -	118
VIII. HYDRAULIC LOSSES - - -	142
IX. AUTOMATICALLY CONTROLLED SMALL WATER-POWER PLANTS -	163
X. REPORTING ON A WATER-POWER PROJECT - - -	184
XI. WATER LEGISLATION - - -	201
XII. HIGH VOLTAGE POWER TRANSMISSION - - -	208
XIII. POWER TRANSMISSION LINE CALCULATIONS - - -	229
XIV. OVERHEAD ELECTRIC POWER TRANSMISSION ECONOMICS -	255
INDEX - - -	267

PRACTICAL WATER-POWER ENGINEERING

CHAPTER I

HYDRAULICS AND WATER-POWER DEVELOPMENT

GOING back to the hydraulic works of the first King of Egypt, "Men of Menes," we are told that the hydraulic works were not only gigantic in themselves, but productive of the most important results to the inhabitants of his kingdom. "Before his time," says Herodotus, "the river flowed entirely along the sandy range of hills which skirt Egypt on the west side. He, however, by banking up the river at the bend which it formed a short distance south of Memphis, laid the ancient channel dry, and dug a new course for the stream half-way between the two lines of hills." The tunnel from the Alban lake is an example of early Roman hydraulic work. This tunnel, of a man's height, and cut through 6,000 ft. of lava, is said to have been begun in obedience to the Delphic oracle in the sixth year of the siege of Vell, B.C. 398. By it the overflow of the lake which used periodically to flood the Campagna was prevented, and the waters were conducted through it in an even flow for the irrigation of the fields which had hitherto lain waste. Another Roman work of still greater importance (great in those days) was the emissarium of Lake Fucino, planned by Julius Cæsar and carried into execution by Claudius. This was a tunnel about 3 miles in length, extending from the lake to the River Liris (the modern Gerigliano), 1 mile of it being cut through a mountain of cornelian rising about 3,000 ft. above the lake. It employed 30,000 men for about eleven years.

In Britain the Romans constructed embankments for the control of rivers, but for at least 1,000 years after their time very little was done in the way of great (great in those days) hydraulic works; and it was not until the beginning of the sixteenth century that the state of the rivers in Italy commanded the attention of the great land-owners and scientific men of that country. At that time, chiefly in consequence of the appointment of a Commission in 1516 by

Francis I., works for remedying existing evils were seriously thought of, and for a long series of years the most eminent mathematicians and engineers of the day were engaged in hydraulic works of greater or less magnitude. About that time the question began to excite interest in England, for it was in the reign of Henry VIII. that a public statute first dealt with river conservancy.

The early methods prevailing in water-power engineering gave no better status to the profession until long after the undershot and overshot waterwheels were looked upon as relics of prehistoric times. In the early days of water-power development their use brought about a rapid expansion in industrial development, until a more economical method of producing power came about in the use of steam power. In those days, in order that the industries might utilise the water power, they were grouped close together where a head-race could be favourably constructed; and, since a head-race for very low fall is, at the best, limited in extent, the industrial development of a water-power community generally took the form of a huddled mass of factory buildings jammed in together as closely as they could be constructed, and the whole industrial works dotted about with shaftings, tunnels, etc. This class of water power afterwards became outgrown and was supplemented with steam plant auxiliary, which latter plant kept growing until the water-power plant was finally used as the auxiliary plant. The most remarkable era came when it was proved beyond doubt that power could be generated in one place and transmitted efficiently over copper wires, mounted on insulators, several miles away. This was closely followed by the evolution of the alternating current motor and the transformer. The generated voltage could then be transformed and the power transmitted and utilised efficiently at the factory or mill, etc., favourably located to suit its own purpose. This remarkable change, coupled with the steady advancement in the art of hydro-electric practice, brought about the development of hydro-electric power on a large scale. Going back only twenty years we find that a 7,000 h.p. turbine was considered to be an exceptional size, but to-day the building of a 40,000 h.p. turbine causes little or no comment. Some of the most noteworthy turbines which have recently been built are:—

70,000 h.p. unit for Niagara Falls,
52,500 h.p. unit for Ontario, Canada,
37,500 h.p. unit for Niagara Falls, U.S.A.,
47,000 h.p. unit for Pit River, California, U.S.A.,
and a 28,000 h.p. unit for a 56-ft. head.

One of the most recent developments embodying the most modern improvements is shown in Fig. 1

It is safe to predict that the future development of the water turbine will be the large size reaction type for high-heads and higher speed units for low-heads. The tendency in this field has already

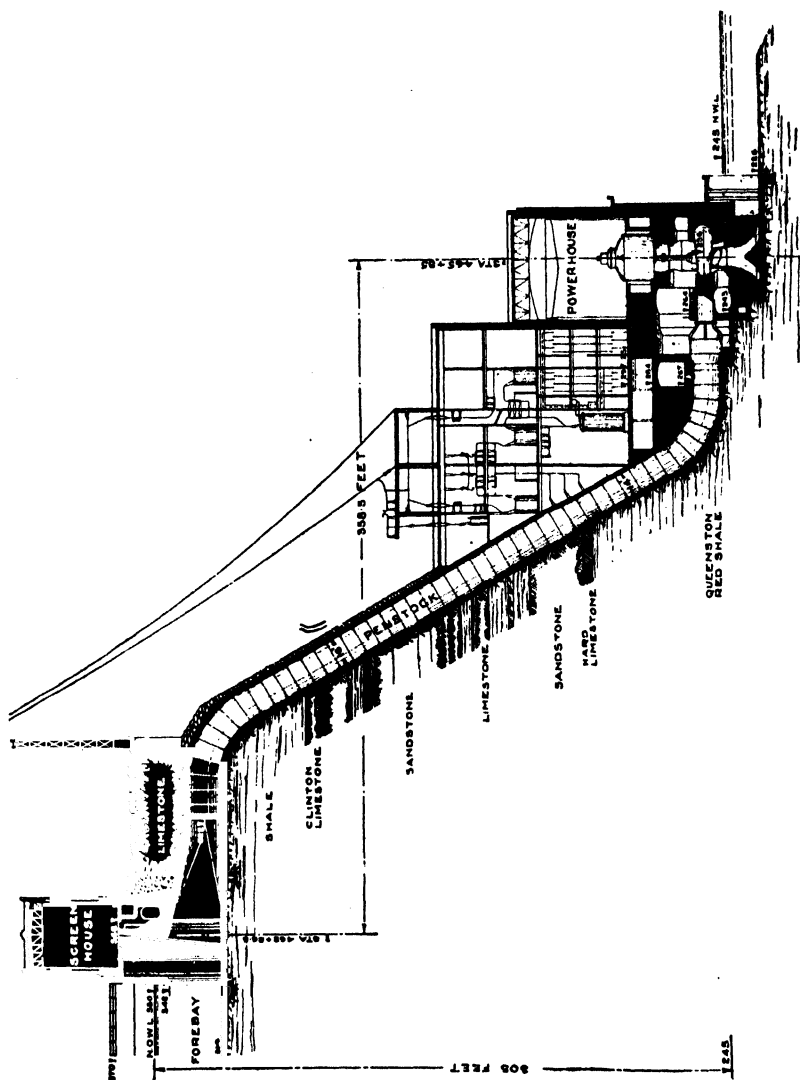


FIG. 1.—This is a Canadian development; one of its principal objects was to secure the highest possible efficiency from all its developed waterways, including conduit, penstocks, and turbines.

been towards larger and larger units of the vertical shaft, single-runner type. This has been made possible by the development of the direct connected hydro-electric unit and the satisfactory solution of the thrust-bearing problem. With this type of unit, operating

under medium- and low-heads, the question of speed is, for commercial reasons, very important. Higher speeds mean smaller and more efficient generators and smaller power station cost. The limit of speed obtainable for the very large units operating under low-heads is determined by the limits of turbine development. Until quite recently the common turbine for low-head plants was of the Francis mixed-flow (inward and outward flow) type. Recently a new type high-speed runner was developed, now known as the "propeller" type. The largest impulse wheel installed is 30,000 h.p.

It is the general practice in the construction of water-power plants to install hydraulic machinery capable of utilising the stream flow much in excess of the absolute minimum and in excess of the flow used in determining the minimum potential water power. This position was forcibly brought out when comparing the minimum potential water power with the total capacity of waterwheels installed in water-power plants existing in U.S.A., as shown in the Reports of the U.S. Geological Survey. The minimum potential water power is based on the average flow of the two seven-day periods of lowest flow in each year of record. This, of course, does not give the absolute minimum flow, but for all practical purposes potential water power based on the flow may be considered as continuous power; the maximum potential water power was based on the flow available for 50 per cent. of the time.

Setting aside for the moment the question of the most economical development, future developments in the way of ordinary and excessive storage—a common class of development which this country can most likely expect—will add considerable importance to the assistance that secondary power will have in the way of saving fuel and operating expenses, etc., during that part of the year when the stream flow is high and, perhaps, when it would otherwise be wasted. In this way the existence of secondary power will permit the closing down of an equivalent amount of steam power. Depending on the character of the development, and more particularly on the amount of storage available, secondary power may be determined by such a condition as the economic balance between the yearly fixed and operating costs of the additional hydraulic and electrical plant, equipment, etc., and the operating expenses necessary with a steam plant of equivalent capacity (see p. 63).

Plants located on streams of different run-off characteristics and storage are generally able to help one another over peak loads and in a large way eliminate steam reserve plant,* water shortage, and other operating troubles. In other words, when power plants are operated in connection with an electrical transmission system, which

* See p. 39.

permits of the following desirable advantages—(a) transfer of load from one river to another ; (b) transfer of load from one plant to another ; (c) transfer of load from one point or locality to another—then, and only then, can each stream be operated at its maximum efficiency. During high-water periods, when there is an excess of water, the water-power units should, whenever possible, be worked at their maximum power capacity (over-all efficiency usually being of secondary importance under these conditions). But, during periods when the amount of water available for power purposes is limited, then the question of operating the plant at its highest possible over-all efficiency is of first importance. In practically all countries there occurs a season with subsequent waste of water, and, in order to make full use of the run-off, it is necessary to use storage and regulate its flow. In the usual way industry seeks only the simplest and most economical projects. Where lakes are not present to offer natural possibilities for effective and relatively cheap regulation of the run-off, it is then necessary to build dams and form artificial reservoirs for impounding the run-off. When water is impounded in this way it has additional value, and high over-all efficiency of the waterways as well as the judicious use of such water (particularly for high-head developments) should be a very valuable and important phase of the operation of such systems. In developments of this class, the stream flow for maximum development is generally based upon the continuous power as indicated by the flow of the stream for, say, five, six, seven, or more months in the year, as determined by averaging the months of each year according to the day of the lowest flow in each, the lowest of the five, six, seven, or more months being taken as the basic month. The average flow of the lowest average of such maximum figures for all years in the period for which information is available is then taken as the estimated flow for the development. Each project will require an independent study to arrive at the best and most economical maximum development point.

The determination of the available power may make necessary the assumption that certain stretches or reaches of the stream or river (as the case may be) form logical development units. In some cases the limits of a power development are unmistakably indicated by the natural conditions ; in others they can be determined only by systematic study, and even then expert engineers working independently often differ as to details.* The fall and water supply are *the* essential factors to be determined. The fall in each development section may be taken from plan and profile surveys. The water supply is computed from the records of stream flow and from a consideration of the probable regulations of storage for the complete development

* See p. 188.

contemplated. A mass diagram is commonly used in determining or/and in studying the effect of storage regulation. The power output can, ordinarily, be estimated at from 60 to 70 per cent. of the theoretical water power. For the ordinary water-power system the total losses and leakage will usually amount to from 30 to 40 per cent. The exact percentage will depend on the type of development, etc.

Floods and low water tend to decrease the power of low-head developments, the former through the lack of head and the latter through lack of water. For alternating current generation, the limit of satisfactory power, in times of flood, is reached when the head of water is so far reduced that the generator speed drops below the limits of permissible frequency. To some degree this has been overcome by the modern design of the single runner (a multiple of runners on the same shaft is no longer installed); also, the application of the "suppressor" has improved conditions of variable head operation. Low water reduces the power nearly in proportion to the amount of water available, but not quite so, for, with very low water the head, in certain low-head developments, can be increased somewhat by reduced elevation of the tail-water, and it is possible to obtain a further increase by the use of flashboards on the dam. With reference to this latter question, the high- and low-water levels to be expected in the tail-water are extremely important. So important are these levels that it might mean in the one case a flooding of the power house and/or a great loss in kilowatt-hour output per annum, and in another case, perhaps complete inoperation. In this matter it would be well to always have in mind the system over-all efficiency, and that the best and most efficient draft-tube design is dependent upon the proper elevation of the turbine. No doubt the simplest way to maintain the normal head and thereby the full capacity of the water-power plant during flood periods (if not too great), is to apply some means for removing the high tail-water.

One of the greatest obstacles to the economic development of low-head water powers has been the great variation of flow characteristics of the rivers. When a water power is developed by damming a stream, the increased depth of water upstream as compared to that below the dam results, in many plants, in a reduction of head during time of high water or flood, due to the greater proportionate rise of level below the dam compared with that above. It follows that at times when a great excess of power is available in the river, the ordinary plant is not only unable to use any of the excess, but its normal output is reduced. Any method which will overcome this difficulty certainly helps to make available those water powers which would otherwise not be economically profitable to develop. Such a method has been provided in the form of a turbine which contains

openings from the turbine-casing into the draft-tube just below the runner, these openings being controlled by a cylinder gate. This is called the "*ejector*" turbine.

Obviously, wherever a stream flows from a higher to a lower level it is possible to develop water power, and the amount of power obtain-

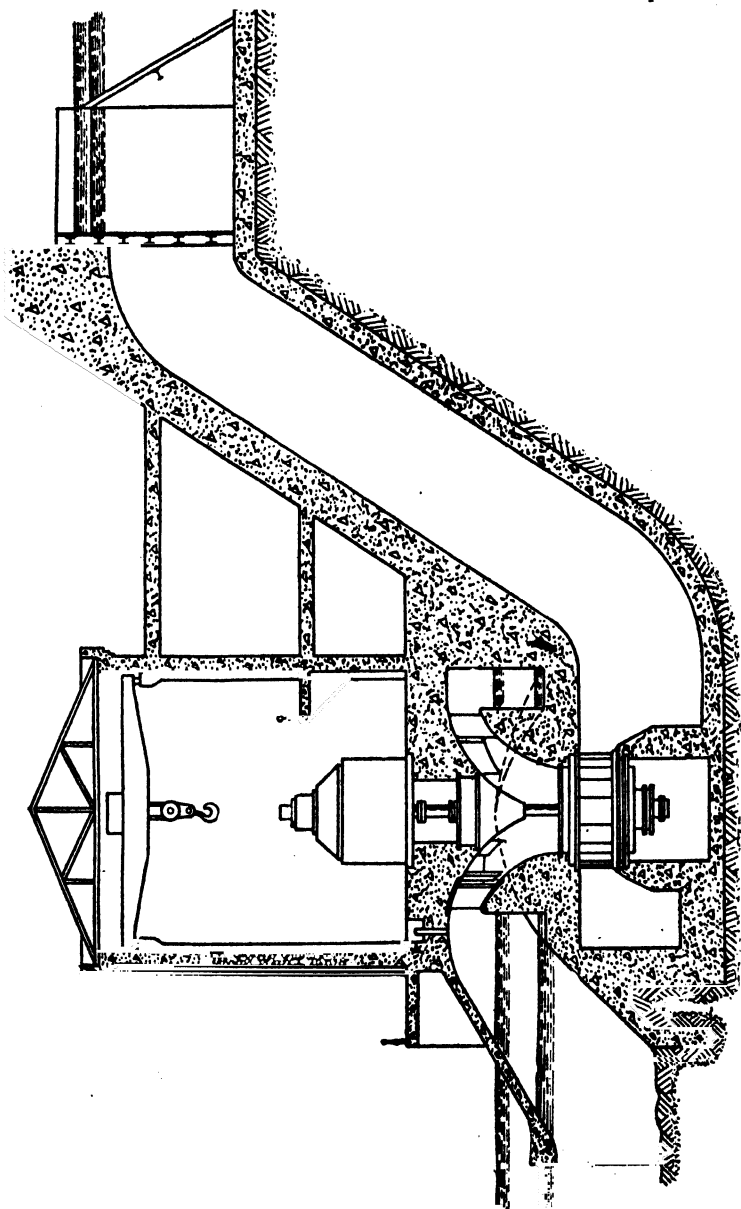


FIG. 1A.—Inverted reaction turbine installation, showing turbine setting below the tail-water level.

able will depend on the supply of water and the available head. Usually in making the choice of a water-power site that part of the stream will be selected where there occurs the most abrupt natural drop or fall on a portion or reach of the stream where there is a well-defined slope. The choice of a water-power site should always be made with due regard to the relative facility and cost of construction. With regard to the quantity of water, it is primarily essential to obtain accurate measurements and also be well assured of each of the following :—

- (a) Minimum quantity of water in dry seasons.
- (b) Average quantity of water available throughout the year.
- (c) The maximum quantity of water for which the waterways and hydraulic works must be provided. In which $Q = \text{ft.}^3/\text{sec.} = \text{cusecs.}$

Now let H' be the total fall of level from the point where the water is taken from the stream or river to the point where it is discharged into it again. Of this total fall a portion is expended in overcoming friction, *i.e.*, the resistances of the various waterways such as the head-race (commonly called the conduit, canal, channel, ditch, etc.), the pipe line (sometimes called the penstock), etc. Let this portion of the head lost by friction equal h ; then the available head for the turbines will be $H = H' - h$. Now let Q be the quantity of water per second; then the *gross* available work of the fall is expressed

$$GQH = 62.4 QH \quad - \quad - \quad - \quad - \quad (\text{ft.-lbs.})$$

Neglecting losses from friction and leakage, the power of this fall may be utilised in several ways. That is to say :—

1. The weight in lbs. ($62.4 Q$) of water may be placed directly on the turbine and arranged to descend in contact with it through the distance H ft. This is by *gravity*.
2. The water may descend in a closed conduit, making the pressure in lbs. per sq. in. at the bottom of the conduit equal to $pQ = 62.4 HQ$. This is by *pressure*.
3. The water may be allowed to acquire velocity

$$V = \sqrt{2gH} = 8.025 \sqrt{H}.$$

The kinetic energy of Q cub. ft. will then be $0.5 GQ(V^2/g) = GHQ$. This is by *inertia*.

$$(H = V^2/2g = 0.0155 V^2); \text{ wherein } g = 32.2.$$

If Q cusecs of water act by weight through a distance h' ft. at a pressure p due to h' of fall, and with a velocity V due to h'' of fall so that $h' + h'' + h''' = H$, then, apart from energy wasted by friction or leakage, etc., the work done will be

$$GQH + pQ + (G/g)Q(V^2/g) \quad - \quad - \quad - \quad (\text{ft.-lbs.})$$

the same as if the water acted simply by its weight while descending H' ft. If the net or effective head H is given in feet and the quantity of water in cusecs, then, for the purpose of preliminary rough calculations, we may assume the efficiency e to be 75 per cent., in which case the amount of power to be derived from a given fall of water is given by the formula—

$$\text{h.p.} = QH \ 0.085.*$$

If the net or effective head is given in feet and the quantity in gallons per minute :—

$$\text{h.p.} = QH \ 0.00023.$$

Water cannot perform its work in the turbine without certain losses of energy, the principal hydraulic sources of loss being: (1) Loss by fluid friction, impact, and eddies; (2) leakage between guide apparatus and wheel and casing; (3) residual velocity in tail-water; (4) loss due to pressure change and speed regulation. The total losses may amount from about 9 to 18 per cent. (making the efficiency 91 to 82 per cent.), according to the size of the turbine. Allowing for bearing friction, the *over-all* efficiency will generally be between 89 to 77 per cent., according to size and circumstances.

In some turbines the whole available energy of the water is converted into kinetic energy before the water acts on the moving part of the turbine. This is the *impulse* type of turbine, and it is set a little above the tail-water and discharges into free air. Hence the water passages of the impulse type of turbine are not completely filled with water. The impulse wheel is usually referred to as a high-pressure turbine or Pelton wheel. The water issues from the nozzle tangentially to the wheel in a solid transparent jet, with a velocity corresponding to the head, reduced only by the friction in the nozzle proper. The water then impinges on double oval-shaped buckets, from which it falls to the bottom of the casing and so away from the wheel-race after doing its work. To secure high efficiency, the jet or jets must not exceed a certain diameter for each diameter of wheel.† For high-pressure turbines with horizontal shafts it is possible to use two, and in extreme cases, three jets per wheel. For vertical shaft wheels, as many as four jets can be used.‡

* The available gross water horse-power is found by the following formula:—

$$\text{h.p.} = \frac{Q \times 62.4 \times \text{net head}}{550} = QH \ 0.1134545 + \quad - \quad - \quad -$$

The brake horse-power is the power delivered at the turbine-shaft and taken from the turbine-shaft; efficiency of 75 per cent. should be taken.

† The ratio of jet diameter to wheel diameter should be not less than about half.

‡ The use of more than two jets per runner will involve the danger of interference of the jets upon the brackets, which nearly always causes a considerable loss in efficiency.

The class of turbine in which part only of the available energy is converted into kinetic energy before the water acts on the turbine is called the *reaction* type. The velocity with which the water enters the turbine is due to the difference of the pressure due to the head and the pressure in the clearance space. In this type of turbine the water passages must be completely filled with water at all times for good efficiency. The turbine is set above or below the tail-water level. Reaction turbines may be placed at any height not exceeding about 28 ft. above the tail-water level. An air-tight draft-tube or suction pipe is employed at the discharge end. In the location of a turbine-runner above tail-water level, the limiting factor is the draft-head or total vacuum at the discharge of the runner. This draft-head is made up of the vertical height from the runner to the tail-race level plus the velocity-head at the runner-throat minus the velocity-head at the outlet of the draft-tube minus the friction loss in the tube. The maximum possible draft-head is around 33 ft. at 70° F. at sea level; but in actual practice it is not advisable to exceed 27 ft. (see H./ d , ratio, p. 162), when the plant is located at somewhat higher elevations than sea level. In high-head plants (where the reaction turbine is now used), to permit of a practical location of the turbine-runner above tail-water, low specific speed runners are employed. On the other hand, it is the practice to use the highest specific speeds (with the resulting advantage of high speed of turbine) for low-head plants. This can be readily understood from the fact that the velocities for low-head are low, and the water will follow the radical curvature of the high specific speed runner; whereas, for high-heads, gradual curvatures must be used to suit the high velocities. For low-head plants, or in other words, high-speed turbines, special attention must be given to design and construction of the draft-tube (assuming, in the first place, that the runner is of correct design), which will regain as much as possible of the energy at the discharge end of the runner. In various parts of the world manufacturers of water turbines have carried out extensive experiments and study of the various features affecting the efficiency of the design of water turbines with the object of reducing the losses in the various parts of the turbine and water passages. Even though the design of the runner is of primary importance, the draft-tube is also of great importance; for no matter how efficient the runner may be in itself, it is possible to lose a considerable percentage of the total head with a design of draft-tube which fails to regain any of the velocity. Experience seems to prove that a long straight draft-tube with a small angle of defusion is the most efficient design. Unfortunately, the long straight draft-tube will require a more costly excavation than the curved type, which, until recently, was considered to be the best to use.

With the larger units the questions of unit spacing and depth of excavation are important factors in determining the exact shape of draft-tube and water passages. The use of draft-tubes of faulty designs and types has probably been responsible to a greater extent than any other single feature for the poor performance of many water-power plants, and due to the faulty design of the water passages from the runner to the tail-race, many thousands of horse-power are being wasted each year. The reduction of efficiency and failure of units to meet the designed capacities, the development of severe vibration, as also impaired regulation, are among the results of poor draft-tube design (see p. 162 for draft-head values).

If the total height of a draft-tube is not well within the theoretical limit of vacuum, the water column will break or the continuity of flow may be interrupted. From the following table it will be seen that it is dependent on the altitude :—

TABLE I.—SHOWING THE BAROMETRIC PRESSURE AND THE EQUIVALENT HEAD OF WATER FOR DIFFERENT ALTITUDES IN MILES

Altitude in Miles.	Equivalent Head of Water.	Barometric Pressure.
	Feet	Lbs. per Sq. In.
0.00	33.95	14.70
0.25	32.38	14.02
0.50	30.79	13.33
0.75	29.24	12.66
1.00	27.76	12.02
1.50	25.13	10.88
2.00	22.83	9.88

Unless actual conditions can be determined it is usually assumed that the tail-water elevation will stand at the same height for a given rate of flow after construction as it does before. This will depend on the layout and stream obstructions caused by the structural works at the site. The elevation is rarely lower, but may be higher.

The rate of flow is generally obtained by the *velocity-area* method which involves the determination of the mean velocity of the water, and, with the average cross-sectional area of the channel known, the rate of flow equals the product ; or by the *direct-discharge* methods, which require the determination of the weight or volume of water flowing in a given time. The gross or total hydrostatic head is obtained by taking the difference in elevation between the head-water surface (lake or artificial reservoir, river, stream, or open channel) and tail-water surface. The net or effective head acting

on a water-power plant is the pressure-head (if a closed conduit, such as a pipe line) or the elevation of the water surface (if an open conduit) at the entrance section plus the velocity head at this point, less the sum of the pressure-head or elevation of the water surface and velocity-head at the discharge section of the plant. In other words, the effective head on the turbine (encased type) is taken as the difference between the elevation corresponding to the pressure in the pipe line near the entrance to the turbine casing, and the elevation of the tail-water at the highest point attained by the discharge from the unit, the above difference being corrected by adding the velocity-head in the pipe line at the point of measurement and subtracting the residual velocity-head at the end of the draft-tube. All pressures are measured from atmospheric pressure to datum. The pressure-head, in feet of water, is equal to $p \div w$, or to $0.016p$; where w is the weight per cubic foot of water, or 62.4 lbs. If p is measured in lbs. per sq. in. the pressure-head in feet is equal to $2.31p$. The velocity-head in the pipe line is taken as the square of the mean velocity V at the point of measurement divided by $2g$, or $V^2/2g$ feet (wherein $g = 32.2$), the mean velocity being equal to the quantity of water flowing in cusecs, divided by the cross-sectional area of the pipe line at the point of measurement in square feet. The residual velocity-head at the end of the draft-tube is taken as V^2 at the end of the draft-tube, divided by $2g$, the value V being equal to the quantity flowing in cusecs, divided by the final cross-sectional discharge area of the closed or submerged portion of the draft-tube in square feet. The velocity of water entering the casing is considered to be effective, and the effective head on the turbine in that case is the pressure-head plus velocity-head at the casing intake plus the difference in elevation between the point of measurement and the end of the draft-tube, less the velocity-head at the latter point. But in the case of an open-flume type the velocity of water coming into the turbine flume is not considered as being effective, and the effective head on the turbine in that case is the difference in elevation between head-water in the flume near the centre of the turbine and the tail-race at the end of the draft-tube subdivision, less the velocity-head at the latter point. In the case of an impulse wheel or turbine without a draft-tube, the effective head on the wheel or turbine is considered to be the pressure-head at the nozzle plus the velocity-head at this point plus the elevation of this point above the tail-water.

The head determines the characteristics of the runner design and the specific speed, which latter is expressed—

$$N_s = \frac{\text{r.p.m.} \times \sqrt{\text{h.p.}}}{H^{5/4}}; \text{ and } \text{r.p.m.} = \frac{N_s(H)^{5/4}}{\sqrt{\text{h.p.}}}$$

which means that the specific speed N_s is the revolutions per minute of a unit at best efficiency if the runner were reduced in size so that it would develop 1 h.p. under 1 ft. of head.*

The factor $H^{5/4} = H \times H^{1/4} = H\sqrt{\sqrt{H}}$.

If the speed is changed in proportion to the square root of the head ($\sqrt{H}$), the horse-power output will change in proportion to the three-halves power of the head ($H^{3/2}$). The rate of flow varies as the square root of the ratio of the effective heads, and as the power output of a turbine is proportional to the product of QH , this output varies as the three-halves power of the ratio of the power output to the power input, and as the input is also proportional to the quantity Q , this output varies as the square root of the ratio of the heads, so that the efficiency varies directly as the ratio of the heads. The efficiency of a hydraulic plant is the ratio of the energy made useful to that which is available. For some parts of the plant the efficiency may be expressed directly in terms of ratio of heads, the losses which occur when there is no leakage being losses of head. For parts of the plant, including the turbine, it can be expressed as a ratio of powers. Where there is a leakage, a correction must be made for the power corresponding to the loss thus sustained. The various efficiencies are defined as:—

1. The efficiency of any part of the plant, not including the turbine, is the ratio of the water power (water horse-power = w.h.p.) at the point of discharge to the water horse-power at the entrance. If there are no leakage losses between these two points, the efficiency may be expressed as a direct ratio between the effective heads at the two points. The efficiency of any part of the plant, including the turbine, is the ratio of the break horse-power delivered by the turbine shaft to the available water horse-power entering that part of the plant for which the efficiency is desired.

2. The plant efficiency is the relation expressed as a ratio of break horse-power (b.h.p.) developed by the turbine to theoretical horse-power (t.h.p.) of the water, expressed:—

$$(b.h.p.)/(t.h.p.) \times 100.$$

The turbine efficiency is expressed:—

$$(b.h.p.)/(w.h.p.) \times 100.$$

The efficiency of any waterway is given by the ratio:—

$$\frac{(w'.h'.p') - (w''.h''.p'')}{(w'.h'.p')}$$

where

$w'.h'.p'$ = water h.p. at entrance ; $w''.h''.p''$ = water h.p. at exit.

* It is now possible to secure specific speeds as high as 110 ft.-lb. units with a decrease in efficiency of not more than 1 per cent. below that attainable at usual speeds, and specific speeds of 150 with a decrease in efficiency of not more than 3 per cent.

The impulse wheel on account of efficiency and design may be limited to a single nozzle on each wheel, and each generating unit be limited to two wheels, one on each end of the shaft. The limiting relation between size of jet, diameter of wheel, and velocity of water, when taken in connection with the other two factors, places a maximum limit to the size of units driven by the impulse wheel. For high heads (about 1,000 ft. and above) the impulse wheel remains the only prime mover, and therefore, where a power station might have large size units, if reaction turbines could be utilised, the use of impulse wheels would materially reduce the size often required. With the use of

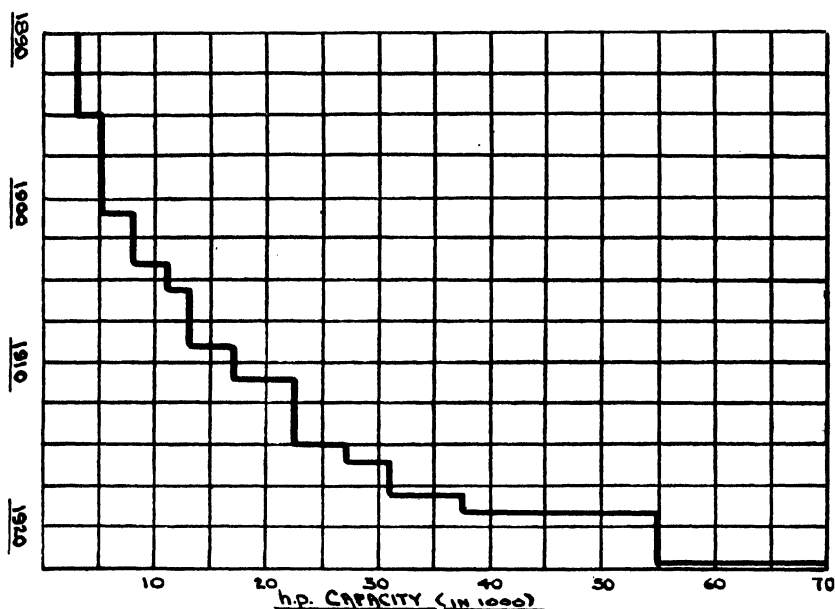


FIG. 2.—Showing development of the water-power turbine.

larger units the reaction type of turbine is being utilised under higher heads, and there is no reason why this type of turbine built with cast-steel scroll-cases should not be used under heads up to 1,000 ft. and possibly higher. So far no limit in size has been reached for reaction type units, the maximum size at present being 70,000 h.p. As units have grown larger, the reaction type may soon, for the larger sizes, practically usurp the domain of the impulse wheel for heads up to about 800 ft.

The vertical shaft units now present a type possessing many advantages over those with horizontal shafts. The principal advantages of this type over the horizontal type are: (a) the turbine-runner is more accessible; (b) a thrust-bearing is not required;

(c) a straight draft-tube is generally possible, giving better discharge and higher efficiency ; (d) the generator can be placed on an upper floor above high water.

Coincident with water-power development may be mentioned improvements in nearly every main part of the complete hydro-electric system. For instance, conduits are now made more permanent, and concrete-lined tunnels are replacing the open canals, ditches, and timber flumes of the past. Valves of the balanced piston type (needle), with the pivot-valves, have replaced the older gate-valves which cannot be used in large sizes under high-heads. The installation of surge-tanks at the junction of conduit and pipe line has become a popular feature. The governor-operated, synchronous bye-pass on impulse wheels has replaced the deflecting nozzle, and, on the reaction type of turbine, the governor-controlled, direct-connected servo-motor is used for operating the turbine gates. The rotating element of the governor is now being placed on the turbine-shaft, thus eliminating belt connection and its dangers. At the power plant and substations all high-tension switches, transformers, and busbars are being placed outside the building, this especially referring to *extra*-high voltage systems. The operating transmission voltage has now been increased to 220,000 volts between line conductors, and, due to the improved state of the art, the limits of overhead power transmission are well beyond the total length of the United Kingdom, *i.e.*, to 1,000 or more miles if commercial conditions will bear the cost.

Respecting water-power development the object, as regards the catchment area, is to obtain at reasonable cost the full utilisation such as maximum possible constant discharge and maximum head. In practice it is sometimes found necessary to forego the full utilisation and use only part of the available head or/and water with a view of developing the total head or/and water at some future date. As regards the developed waterways, the object is to obtain the most economical discharge and highest net head at the turbine ; and to install the best types of plant, etc., best adapted to maintain close-speed regulation and highest efficiencies. The largest and fewest number of units of higher efficiency are now installed, and the more economical power systems are now linked and worked at high load factor and in relation with the shape of the load curve.

With water-power developments actual experience of the right kind is the best guide. And, as with men—so with the developed waterways, water turbines, etc.—it is the last few per cent. that distinguishes.

CHAPTER II

CATCHMENT, RAINFALL, AND RUN-OFF

THE extent of stream flow from a catchment area depends primarily upon the total area of the basin drained, the depth of rainfall (total precipitation, as the elements of rain, snow, etc.), the geological formation, the topographic features, as also on the general characteristics of the area. That the catchment areas of equivalent size give little relationship as to the actual run-off or stream flow is undeniable, because the catchments may vary, and do vary, to the extent of two opposite extremes, as, for instance, from glaciers and snowfields of mountain ranges to the arid region of a dry belt. Hence the same conditions of power development cannot be expected from two streams differently situated, etc. The average time-period of peak flood will also depend largely upon the topography and upon the elevations within the catchments and the mean latitude of these catchments. Where the catchment includes a large area of high mountains, some of the winter precipitation may be stored in snow, which later in the season contributes to the spring freshet. The greatest floods of the year (in certain countries) occur when a spell of warmer weather, combined with heavy rain, causes excessive melting of the snow. It is recognised that neither catchment areas nor water areas, nor, in general, average differences of elevation over considerable distances have, necessarily, any specially significant bearing upon estimates of the amount of available water power. That is, the presence of natural reservoirs do not necessarily imply the existence of considerable water power. Extensive water areas may, nevertheless, provide good reservoirs where water may be effectively impounded, and, indirectly, water is generally of much greater value to a water power receiving direct benefit from such storage than water power without storage. In all cases, if we desire to turn these extensive water areas into producers of power, head is required.

In physical geography the catchment or drainage area is commonly defined as the boundary separating the head-streams tributary to two different river systems or basins (see Fig. 3). The crest of a mountain ridge forms the most clearly marked boundary. In a plain country of gentle slope the boundary line is often difficult to trace,

as the head-waters of two different river systems may merge in marshes or lakes at the highest levels. A stream or river is the channel for the discharge of the available rainfall on the catchment area. An approximate relation of the average stream discharge to the average rainfall on the catchment is :—

TABLE II.—SHOWING THE APPROXIMATE RELATION OF THE AVERAGE DISCHARGE TO THE AVERAGE RAINFALL

Condition of Ground.	Ratio of Average Discharge to Average Rainfall.	Loss by Evaporation, etc., in percentage of Total Rainfall.
Cultivated land - - - -	0.30 to 0.33	67 to 70 per cent.
Wooded hilly slopes - - -	0.35 „ 0.45	55 „ 65 „
Bare unfissured hills - - -	0.55 „ 0.60	40 „ 45 „

The shape and area of a catchment (with reference to the surface only) may not be coincident with that of the corresponding underground catchment, as the structure of rocks largely determines the direction of flow of the rainfall as sinks into the ground. Furthermore, the flow in any watercourse (especially for the smaller catchment areas) may, by reason of underground flow into other catchments, be disproportionate to the area apparently drained by the watercourse.

Fig. 3 illustrates a very favourable position of a natural lake in relation to the total catchment area. Water from the entire catchment area of this river is stored. If, however, this lake had been located at A, B, or C, the catchment area as shown could not be made available for drainage and catchment of the same amount of water. As indicated in Fig. 3, it may also be possible, depending

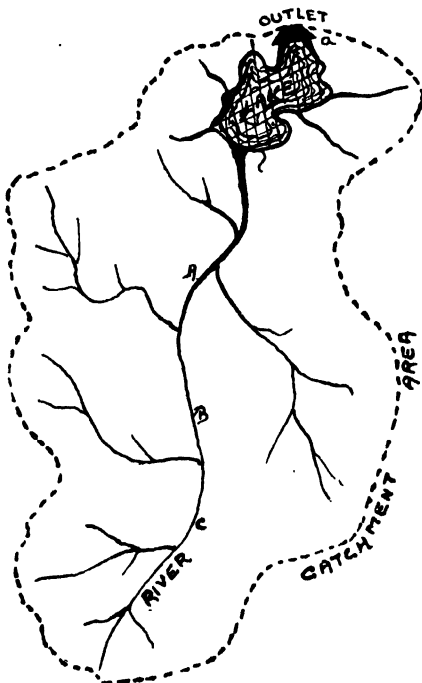


FIG. 3.—Typical catchment area showing possible sites for water-power development.

on the topography, to develop power economically at each of the three points A, B, or C on the river, and at the same time develop full power at *a* from the stored water drained from the entire catchment area. Moreover, if lakes had been located at A, B, and C respectively, the respective catchment areas, as also the respective storage capacities, would have to be treated independently.

It is generally accepted that forests receive a greater quantity of rain, snow, and hail than open land in the same vicinity (see p. 21); that mountainous areas, whether bare or covered with forest growth, receive more rain than flat and low-lying country; that forests in mountainous areas receive more rain than open land at the same elevation; that the effect of mountainous districts is to cause the rain which falls on them to be more equally distributed, both throughout the seasons and from year to year; and that a local excessive rainfall may probably be due chiefly to a precipitous face of a projection from high mountains which tends to arrest and condense the current of moist air. The conservation of water supplies is inseparably connected with that of *forests*.

While records of run-off from a catchment area are of first importance in the question of water-power studies and development, the rainfall is also of extreme value, in that these latter records—if of a more extensive period than those of the run-off—would indicate the high and low range of stream flow which might be expected. It is probable that an additional check may be had from an examination of the annual rings of good-sized trees in the catchment. At the present time it is taken that rainfall records of a catchment area, in which no discharge measurements are available, can sometimes be used for the estimation of the stream flow based on the rainfall and run-off records of an adjacent catchment of approximately similar dimensions and characteristics. It is known that comparative studies of rainfall for the years during and previous to the years of stream flow record will often indicate, with much accuracy, the possibility of great variation from the recorded average flow of any stream.

It is often found (but not always) that the mean rainfall over a long period can be determined, for an area upon which the total rainfall is known only for a short period, by assigning to the missing years of the short-period gauges rainfalls bearing the same proportion to those of corresponding years in the long-period gauges that the rainfalls of the known years in the short-period gauges bear to those of corresponding years in the long-period gauges. If we can judge from a long list of existing records already available, it would appear that at any given place the total rainfall during any period of fifty years will be within about 2 per cent. of the total rainfall at

the same place during any other period of fifty years, while for records of any period of twenty-five years it has been found to fall within about 3.25 per cent. of the mean of fifty years. It is now recognised that there is a nearly constant ratio on any given area, exceeding about 1,000 acres, between the true mean annual rainfall, the rainfall of the driest year, the two driest consecutive years, and any other group of driest consecutive years. Thus in any period of fifty years the driest year upon such an area can be taken to be *about* as follows :—

TABLE III.—SHOWING APPROXIMATE DRIEST CONSECUTIVE YEARS OF RAINFALL FOR GIVEN AREA

	Percentage of the Mean for the 50-Year Period.
In the <i>two</i> driest consecutive years — — —	75 per cent.
In the <i>three</i> driest consecutive years — — —	80 „
In the <i>four</i> driest consecutive years — — —	83 „
In the <i>five</i> driest consecutive years — — —	85 „

Calculations for available power depend primarily, not upon total precipitation or rainfall, although this is the original source, but upon the flow of the particular stream under investigation. If the daily flow of the stream is known for a long and continuous period or for a period of, say, seven, ten, or more years, which includes the severest drought, it would be possible (by plotting these records) to lay down a line of power demand representing a quantity which it is desired to obtain from the catchment area and to scale off directly the approximate power and the storage necessary to provide for that demand which would most likely represent the estimated flow for maximum development or best development point. The more reliable basis would be to always deal with stream gauging in preference to collection of rainfall data, as stream gauging is always very much more important and more reliable than rain gauging. As a matter of fact, it is from the discharge of the stream that the available quantity of water is found, not from the total precipitation (rain, hail, and snow) which is naturally influenced by all sorts of complexed conditions before it actually reaches the streams. Moreover, the relative run-off is made subject to a coefficient value dependent very largely on the “personal equation.” Leaving this aside, the investigating engineer is particularly interested in the rate at which the run-off occurs and in the quantity thereof in a time-period, *i.e.*, given percentage of time for given amount.

Although it is found quite practical to establish a useful relation (expressing *probability*) between the run-off and the rainfall, it is quite impossible to define or fix a law which can be relied upon to give this relation with accuracy for each year and to suit every size and character of catchment. For instance, if we consider only the same aggregate for one year in the same area, we may find that the rainfall has been concentrated in a few storms of great intensity, while in another year the rainfall has been distributed over a great number of storms with barely enough rain to saturate the surface soil, and with sufficient intervals between storms to permit the soil to lose its water by evaporation. Thus, in the former case a relatively large proportion of the rain will find its way into the stream, whereas in the latter case very little if any run-off will occur, although approximately the same amount of rainfall may be recorded. Further, it is impossible for anyone to predict the rainfall as to time and quantity for future years except on the basis of past records, and it is principally with this limitation on *estimated* water yield and *estimated* maximum and minimum flow that the relation of run-off to rainfall should be studied. Furthermore, in the study of rainfall intensity applying to any catchment, it follows that even in the case of the very best located rain-gauge stations only an approximation of the actual rainfall can be recorded. Some of the reasons for this latter are: (1) Each rain-gauge record is only an approximation of what occurs on and, perhaps, a few yards away from the station; (2) maximum rates of rainfall during a storm at various points on a catchment do not occur at exactly the same time; (3) distribution of rainfall is not the same in all places nor for all storms; (4) a rain gauge records only what is happening at the particular point where it is placed; (5) the maximum possible storm may not be among the particular storms which have been recorded, etc. For catchment areas of considerable extent, the relation of run-off to rainfall is further complicated by the difficulty always encountered in the determination of total precipitation as well as by the lack of precision in the estimates of stream flow. The principal elements of uncertainty are snow, hail, rain, and so forth, and only the crudest kind of approximation is often resorted to—the personal equation perhaps again predominating in regard to the choice of a reliable coefficient, etc. It is suggested that, in order to get a more reliable estimate of the annual precipitation upon a catchment area by the ordinary rain-gauge method, there should be one rain gauge to about every 0.75 sq. mile or 1.0 sq. mile, and that these rain gauges be properly located.

The relation of precipitation to area should be determined from a consideration of all storms showing average frequencies of recur-

rence of a given period, even though the central gauge did not show the maximum recorded precipitation; and that the average precipitation should be determined over areas of given size having their centre at the central gauge. About the best arrangement is a central control gauge and several others located about equally distant from it and preferably not more than one mile away.

In the case of a single mountain valley which is to be used as a storage reservoir it would no doubt be necessary to have records of rainfall at many points; at least three being in the axis of the valley, one of which is near the point of intersection of the axis with the boundary of the catchment. Also, in order to connect with these the effect of the right and left slopes, there might be at least one gauge on each side about the middle height, and approximately in line vertical to the axis of the valley passing through the central gauge. The relative depth recorded in the several gauges would depend mainly upon the direction of the valley and steepness of the boundary hills. Of course, much will depend on the judicious placing of the gauges. Each gauge should have from 40 to 60 ft. around it, also an uninterrupted plane fairly representing the general inclination of the ground for a much larger distance around it. A central gauge should be useful for correcting and checking the other gauges. If the gauges are properly placed, the arithmetical mean of their results will probably not differ widely from the true mean rainfall of the valley.

The value of forest and shrub in condensing moisture and the effect of irrigation with its crops in producing or slightly increasing rainfall is not generally disputed (see p. 18). The air over ground covered with forest, shrub, or green crops—protected from the direct rays of the sun—is much cooler than it is over country not so covered, and therefore is able to condense the heated and saturated air, perhaps resulting in more steady rainfall. The heated air over bare country cannot condense the moist air, which rises, causing violent downpours of rain, finally resulting in some of the best sloping land being stripped of its soil. In China, India, and other parts of the world, vast areas of the uplands are quite denuded of soil owing to the lack of covered ground and the consequent destructive storms. A scoured and deforested area which has lost the humus of its soil cannot hold the rains which, instead of being retained in the ground and transmitted into plants by the various processes of growth, carry destruction in their pathways. Investigations apparently have proved that, with the introduction of forest growth, rainfall that has ceased may be made to start again and, by hastening most kinds of tree growth on desirable catchment areas, it is very likely that rainfall will be increased and be made more regular (see p. 201).

Evaporation from the soil surface is dependent upon the same factors which control evaporation from water surface, and, in addition, it is dependent on the degree of saturation of the soil surface. The amount of evaporation from water surfaces is mainly controlled by temperature, humidity, and wind velocity. Wind velocity generally tends to increase with elevation. With a large surface area the total evaporation will be slightly greater in proportion to that for a small surface area. The direct evaporation loss during heavy rains is usually only a very small proportion of the available water supply. One of the important factors affecting evaporation losses of all kinds is temperature. As the result of experiments on evaporation, it has been found that evaporation from flowing water is about 8 per cent. greater than from still water, all other factors being equal, and that an increase in water temperature produces practically corresponding increases in evaporation. Temperature is also an important factor in relation to floods as affecting the rate of melting of accumulated snow. According to locality and climate, evaporation per annum from the water surface of reservoirs varies from about 2 to 10 ft.—6 ft. was recorded by the writer during his studies in East Africa.

If the rainfall for a given year is very little the time during which the soil is wet and evaporation rapid will be short; consequently, in such years evaporation will take less of the rain water than it will in wet years. If, however, the number of rainy days is great, and the conditions favouring evaporation are poor (by reason of the long time during which the atmosphere is saturated), then, even with a saturated condition of surface soils, there may be less evaporation than would possibly occur for a fairly regular rainy period of a few days with a dry period of similar duration. Evaporation may therefore take more of the falling water as rain in fairly wet years than in dry years, and the evaporation may take from the soil practically all the water which falls in dry years, when the rainfall is very little, perhaps leaving nothing to go to the stream.

The *probable* ratio of evaporation to rainfall will decrease as the quantity of rain increases up to some more or less definite limit, and, conversely, the proportion of rain water which reaches the stream as run-off will increase. In the study of stream flow and run-off in a catchment area it is very necessary to look carefully into the relative proportion of water surface formed by lakes, storage, pondage, and the like, and the variation of water surface from one season or one year to the next; this will usually involve the study of loss of water by evaporation. In such like catchment areas with a large proportion of water surface, it may be found that when the rainfall is little, the loss of water by evaporation exceeds the accession resulting from rain falling directly on the water. The water production from such an

area (catchment embracing lakes, etc.) of considerable extent may therefore be a *negative* value.

It certainly is evident that the extent of run-off—surface flow or stream flow—depends principally upon the depth of rainfall and the area and character of the catchment area or basin drained; that is to say, it depends on such features as the geological formation and topography of the catchment area above the power site; proportion of forested, watered, and bare area; contour of the ground; nature of ground; altitude; climatic conditions, etc. Therefore, the run-off of streams can be expected to vary not only from season to season, but also to such an extent from year to year that the same conditions are not likely to occur in any two successive years. Records for a cycle of *at least* seven years are necessary to cover the yearly variation to be anticipated, but records extending for a period of forty or more years would certainly be more reliable, although, in rare cases, it has been found that a cycle of five years has given sufficiently reliable results to warrant a power development design.

In the determination of the *probable* run-off for different periods and typical years for a given power site, much will depend upon the character of the available records. Unfortunately at the present time few records are what one might call complete. In the determination of the most probable run-off a great deal must be left to the judgment of the investigating engineer. It might be found, upon investigation, that no run-off records are to be found, but ample rainfall records of an adjacent catchment area are available. Or there may be no run-off records of the particular power site, but numerous run-off records at stations in the same or in adjacent catchments of similar characteristics. Failing stream flow measurements, the most obvious requirement is to have long-period and continuous run-off records for the actual power site under consideration. Discharge measurements are usually computed—in their final form—in terms of run-off in cusecs or cusecs per square mile of catchment area. It is important that this latter, the area of each catchment, be obtained as accurately as possible. At the present time there is little excuse for not obtaining correct catchment area dimensions, as excellent Ordnance and other maps are now made available to all at little cost.

In terms of *average* values of rainfall, catchment area, and head, the available power can be expressed :—

$$\begin{aligned} \text{h.p.} &= 0.00828 \, fmH, \\ \text{e.h.p.} &= 0.0073 \, fmH, \\ \text{kW.} &= 0.0062 \, fmH, \end{aligned}$$

wherein f = average rainfall in inches.

m = catchment area in square miles.

H = average head in feet.

For example: The catchment of the Lochaber water-power scheme is 303 sq. miles; the average annual rainfall is 73 in.; and the average head of water available near Fort William is 742 ft. What will be the theoretical water power and energy?

$$\text{h.p.} = 0.00828 \times 73 \times 742 = 136,000 \text{ h.p.,}$$

and, as 1.0 h.p.-year = $1 \times 8,760 \times 0.746 = 6,535$ kW.-hours, the theoretical output would be

$$6,535 \times 136,000 = 888.76 \text{ million kW.-hours.}$$

The most usual way of calculating the run-off efficiency of the catchment is to divide it into areas on which the rainfall is of uniform amount, and the slopes and general character of the surface are the same. In the measurement of the catchment area it is customary to work it out on the topographical map and to planimeter the enclosed area or to mark it out by ruling squares to scale. The average yield of each constituent area can then be estimated and the total calculated average yield would then be equal to their sum.

The Simpson rule may also be used as a check, that is to say, divide the length of the figure into any even number of equal parts, at the common distance L apart, and draw ordinates through the points of division to touch the boundary lines. Add together the first and last ordinates and call the sum x ; add together the even ordinates and call the sum y ; add together the odd ordinates, except the first and last, and call the sum z ; then the area of the figure is

$$A^{\circ} = [L(x + 4y + 2z)] \div 2.$$

Another simple rule, called the *trapezoidal* rule, is to divide the figure into a sufficient number of equal parts; add half the sum of the two end ordinates to the sum of all the other ordinates; divide by the number of spaces (that is, one less than the number of ordinates) to obtain the mean ordinate, and multiply this by the length to obtain the area.

The method to use for a determination of *probable* run-off for a given area (taken at the power site) will depend largely upon the character of the available records, such as long-period or short-period or broken-period records, as the case might be. They will depend on records of the area taken at the power site under consideration, or for an adjacent catchment or parts of different catchments or watersheds, and rainfall records in the area or within reasonable distance and at the same or perhaps at a different altitude, etc. Some of the methods applicable to the determination of probable run-off are:—

1. Arrangement of the stream flow records by months in order of dryness, regardless of the year in which they occur, then compute the probable run-off for an *average* year.

2. Similar to the above (1), except that average weekly or daily twenty-four hour rate of flow can be used in place of the monthly records.

3. Preparation of daily, weekly, or monthly records in tabular form arranged in order of years, but with the days, weeks, or months arranged in order of dryness; the driest periods are averaged, then the *second* driest periods, then follow the *third* driest periods, and so on, and a computed *average* year is finally obtained.

4. Preparation of a table with rates of flow in cusecs or cusecs per square mile, arranged in order of years, months, weeks, or days, and a final computation of the *average* discharge for each period (month, week, or day) as desired.

5. Tabulation of the discharges or corresponding gauge heights, marking for each year of record the total number of days when the flow or gauge height is not greater than the given or desired amount. The run-off for an *average* year is obtained by adding the total number of days for any number of years, and finally taking the *mean* amount.

6. Arrangement of the rates of discharge in order of magnitude for each year, taking the mean value of the maxima, then the *next* lower amount; then follows the *third* lower, and so on down to the desired or given smallest amount; finally, a complete *average* year is obtained.

7. Preparation of a table showing the number of times various rates of flow have occurred during the period of record. The final results are expressed in terms of percentage values to serve an *average* year, showing the range from the extreme maximum to the minimum rates of flow.

The desired or given amount of flow without storage is usually that of the *ordinary* minimum flow, which amount is ordinarily based on the averages of the minimum flow for the *lowest two* consecutive seven-day periods in each year, over the period for which records are available.*

For the completion of power estimates, it is becoming the practice to give the *estimated* flow for maximum development, stating how the calculation is made—whether for six, seven, or nine months' flow on the basis of continuous power at a given efficiency. At the present time a six months' period is usually taken, that is—

8. Arrange the months of each year according to the day of the lowest flow in each—the lowest of the six *high* months being taken as the basic month. The *average* flow of the lowest seven consecutive days in this basic month will determine the maximum for that year.

* Throughout the Dominion of Canada power estimates are based on this amount for twenty-four hour power at 80 per cent. efficiency at the power house.

The *averages* of such maxima for all years in the period (for which records are available) is the estimated flow for maximum development.

To be reasonably reliable, formulæ for rainfall must take into account the *maximum* rainfall, intensity, and time period, slope, and nature of the country drained, as well as other considerations. The run-off is more often expressed as a percentage of the rainfall; that is, by

$$r' = \text{run-off} = f - E / f \ 100 \quad - \quad - \quad - \quad (\text{per cent.})$$

wherein f = rainfall in inches.

E = evaporation and seepage losses in inches.

Considering the factor of run-off only, this factor is influenced by so many variable elements that it is not yet possible to give a general formula that will satisfy *all* conditions; in other words, each catchment and each stream is a law by itself. To express the relationship between rainfall and run-off, or the run-off itself, the following methods have been proposed:—

1. Set out in tabular form or preferably plot curves in terms of a percentage of the total annual run-off, showing the weekly, monthly, or quarterly run-off (as required) to be expected from the given catchment.

2. By taking 23 in. of rainfall as the amount required to supply the total losses; or

(2A) By taking 12 to 18 in. as the amount.

3. By preparing curves of an exponential formulæ, showing the relationship for storage, growing, and replenishing periods.

4. Use of annual run-off by an equation consisting of a coefficient, *multiplied* by the square of the annual rainfall; the coefficient varying with slope and mean annual temperature for different catchments.

5. By taking

$$r'' = 0.07 f^2 \quad (\text{for } f \text{ less than } 50 \text{ in.}),$$

and

$$r'' = f - 0.25 \quad (\text{for } f \text{ greater than } 50 \text{ in.}),$$

wherein

f = total annual rainfall or precipitation in inches on any catchment.

r'' = run-off depth in inches over the same area, resulting from this precipitation.

6. Use of a constant *plus* a percentage for the several months of the year, varying the relationship on different catchments with the mean annual temperature.

7. By plotting two *typical* curves, one for streams in mountainous regions, and the other for streams draining from broad valleys with gentle slopes.

8. By taking actual dry-weather flow at, say, 0.25 cusec per 1,000 acres (1.56 sq. miles) of catchment.

9. By taking annual run-off in cubic feet yielded from the catchment as $3.63 A (f - E)$; wherein f = rainfall of a given area in inches; E = losses by evaporation and seepage during the year in inches; M = area of catchment in acres.

10. By taking run-off in cusecs yielded from the whole catchment area, as $Q = 200 m^{5/6}$; wherein m = area of catchment in square miles.

11. By taking run-off in cusecs yielded from the whole catchment area, as $Q = cm^{3/4}$; wherein m = area of catchment in square miles; $c = 300$ to 350 for 6 in. rainfall, 300 for hilly country for a rainfall of 3.5 to 4 in.; and 250 in mixed country, and 200 in flat country.

12. By taking run-off in cusecs yielded from the whole catchment area, as $Q = 1,300 (m/l^{2/3})$; wherein m = area of catchment in square miles; l = length of catchment in miles; where Q , in the above, is the *maximum* run-off.

Climatic conditions, such as humidity, temperature, wind, and frequency of thunderstorms; topographic features, such as mountains, valleys, and lakes; geographic position, such as proximity to the ocean with its varying currents; location in the paths to the windward or leeward of mountain ranges, and the like—all these factors exert a great influence on the frequency of and the intensity of precipitation. There is a general belief that the records of a number of rain-gauge stations in a region of a somewhat similar climate and topographical features can be combined to produce an equivalent long-period record that is as good an indication of the frequency of given rates of intense precipitation as records of the given region.

Judging from these different formulæ and proposals, it is seen that in general they were devised to represent run-off conditions at some particular locality and within certain limiting ranges of slope, area, rainfall, and imperviousness. These different proposals do not include all the formulæ, etc., so far proposed. Nevertheless, when all are brought together and a deduction made, it is seen that there still exists a great need for a formula to suit each clearly defined set of conditions, or a formula of general application which can be used in a *new* locality with some reasonable assurance that it will be fairly reliable, will be applicable for large or small areas, flat or steep slopes, heavy or light rainfall, and imperviousness of ground.

Examples.—Assume that, in a month of thirty-one days, the rainfall on a catchment area is 10 in., and 60 per cent. of the rainfall (6 in.) is estimated to run off in that time; then the average run-off will be 5.21 cusecs. That is to say, if the entire rainfall is run off, then 1 in. depth in thirty-one days of twenty-four hours each will equal

0.868 cusec, and 10 in. will discharge 8.68 cusecs ; 60 per cent. of this amount will make 5.21 cusecs run off per square mile during the month. It will also represent a discharge of 13.94 million cub. ft. per sq. mile, or 320 acre-ft. per sq. mile, or 10.4 acre-ft. per day per sq. mile for a month of thirty-one days. From the equivalent factors we have

$$2.678 \times \text{cusecs per sq. mile during the period} = 2.678 \times 5.21 \\ = 13.94 \text{ million cub. ft. per sq. mile,}$$

or

$$61.49/\text{acres per sq. mile} = 61.49/640 = 10.4 \text{ acre-ft. per day} \\ \text{per sq. mile (for period) ;}$$

but

$$53.3 \times \text{actual run-off} = 53.3 \times 6 = 320 \text{ acre-ft. per sq. mile,}$$

and

$$0.538453 \times \text{cusecs} \times \text{days} = 0.538453 \times 5.21 \times 31 = 86,965 \text{ million gals.} \\ \text{discharged ;}$$

also, as before,

$$86,965 \times \text{acre-ft. per million gals.} = 86,965 \times 3.68 \\ = 320 \text{ acre-ft. per sq. mile.}$$

As every cusec of water falling 88 ft. equals 100 h.p., every square mile of this catchment area is equal to 521 h.p., theoretical horse-power during the given period of time, working under such a fall.

Where records have been taken, it will usually involve a checking of the rate of flow with rainfall and run-off. Taking an *example* : records show that the total rainfall from, say, 6 P.M., 4th November 1922 to midnight, 7th November 1922, was estimated at 7 in., and of this rainfall about 70 per cent., or about 5.0 in., flowed off ; or an equivalent of $26.9/\text{days} = 26.9/3.5 = 7.7$ cusecs per sq. mile per in. depth of rainfall. Records also show that the maximum rain in twenty-four hours was about 3 in., and the maximum rate of flow was 1.5 in. per twenty-four hours, or one-half that of the rainfall. This maximum rate of flow is

$$Q' = 1.5 \times 26.9 = 40.35 \text{ cusecs per day per sq. mile.}$$

The size of the catchment (planimetered from map) is about 80 sq. miles, and the topography will approximate, say, an average for a neighbouring area. The total run-off is

$$40.35 \times 80 = 3,228 \text{ cusecs from the catchment.}$$

On a neighbouring stream of 30 sq. miles of catchment the maximum rate of flow is estimated at 1.6 in. in twenty-four hours, or

$$1.6 \times 26.9 = 40.03 \text{ cusecs per day per sq. mile,}$$

or

$$40.03 \times 30 = 1,291 \text{ total run-off in cusecs from catchment}$$

In this particular case the two rates of flow are approximately the same, which shows that the heavy rainfall was of sufficient duration to affect fully the larger catchment area.

The average annual rainfall at any given place should preferably be deduced from observations extending over thirty or more years; these will give the mean annual fall, probably correct within a few per cent. For observations extending over shorter periods, some engineers maintain that the error in per cent. is *about* as follows:—

Number of Years.	Error in per cent.
25	3.0
20	4.0
15	5.5
10	8.0
5	15.0

The ratio of the wettest year to the mean annual rainfall may average about 1.55; the probable rainfall at any given place in the driest year may be taken at about 0.65 of the mean annual rainfall; the percentage of the mean annual rainfall for a 50-year period is about 75 per cent. for the two driest consecutive years.

Useful Conversions

Catchments.—

1 in. of rain = 3,630 cub. ft. per acre.

= 100 tons per acre.

= 22,000 gals.

= 64,000 tons.

= 2.325 million cub. ft. per sq. mile.

1 ft. depth on 100 sq. miles = 2.79 billions cub. ft.

= 3,227 cusecs for ten days.

= 1,076 cusecs for thirty days.

= 88 cusecs for one year.

Inches of rainfall $\times 14.5$ = million gals. per sq. mile.

1 in. depth over 1.0 sq. mile = 2.325 million cub. ft. yearly.

„ of rainfall per month = 0.9 cusec per sq. mile.

„ of run-off per year = 0.0736 „ „ „

„ of run-off per hour = 640.0 „ „ „

„ „ „ = 1.0 cusec per acre.

1 cusec per sq. mile = 13.56 in. yearly run-off.

„ „ „ = 31.54 million cub. ft. yearly.

Rainfall of South and East counties is less than West and Midland, but nearly as many rainy days. Going from West to East across England, rainfall increases as the hill ranges are approached, then decreases and reaches a minimum in the Eastern counties between the Humber and the Thames. Series of three or even four consecutive dry years occur and must be allowed for. Wettest years,

one-third to one-half as much again as average ; driest, one-third less ; mean of driest two consecutive years is about one-fourth less.

AVERAGE ANNUAL RAINFALL

Great Britain and Wales	-	-	-	-	34 in.
England	-	-	-	-	31.91 in.
Wales	-	-	-	-	46.15 "
Scotland	-	-	-	-	44.85 "
Ireland	-	-	-	-	44.86 "

Deep-seated springs and wells receive from about .3 to .4 total rainfall in this country.

Average daily evaporation from reservoirs is about .07 in. for England.

APPROXIMATE QUANTITY OF WATER ABSORBED BY SOILS

Material.	Volume of Water Absorbed per 100 of Material.				
Sand	-	-	-	-	30 to 40
Coarse sand	-	-	-	-	39.5
Sandy soil	-	-	-	-	45.5
Chalk soil	-	-	-	-	49.5
Clay	-	-	-	-	50 to 53
Loam	-	-	-	-	45 " 60
Peat subsoil	-	-	-	-	84

POROSITY OF ROCK

Material.	Porosity in Percentage.				
Granite	-	-	-	-	0.12 to 0.25
Dolarite	-	-	-	-	0.24
Marble	-	-	-	-	0.75 to 0.95
Compact quartzite	-	-	-	-	2.9
Fine-grained sandstone	-	-	-	-	12 to 17
Compact limestone	-	-	-	-	17 " 20

CHAPTER III

SELECTION OF WATER-POWER SITES

ONE of the most important phases in the study of water power is in the selection of the sites, *i.e.*, dam, conduit, forebay, pipe line, power house, and tail-race. Each site is governed to a large extent by the geological and topographical conditions. Where storage is available, its four main controlling factors, each involving many variables, are the reservoir, bottom, head, and total cost. The location, arrangement, and cost of the head-works are governed almost entirely by natural conditions. The best location of the forebay is usually a matter of cost, but with favourable geologic and topographic conditions, the most effective combination of forebay-regulating basin should be made at relatively low cost. The location of the pipe line requires good hill side, good slope, and relatively cheap and ample means for supports and anchorage, etc. The location of a conduit (non-pressure type) depends essentially on local conditions. Power-house location demands good foundation and the best use of head to give maximum power and efficiency at all stages of the river.

As the economical development of any water power depends upon the continuous available power from the river and upon the market for power, it follows that two of the main conditions to be determined as accurately as possible to bring about the desired economic results are, first, a reliable analysis of the hydrographic conditions, and secondly, a reliable estimate of the load factor and character of the load. Studies with reference to power output can be made more useful by plotting the information in the form of duration curves, which will show the duration of each intensity of flow for each year without reference to the day of occurrence. From all the information available an *average* duration curve can be plotted from all the records obtainable. From the average duration curve the average quantities of power that could have been developed from the stream during the period of record can be determined (see p. 52).

As regards the present status of water powers, with particular reference to studies, records, the collating and analysis of hydrographic and hydrometric data and conditions, one is not very wide of the mark in saying that even at this stage of the art the engineer

PRACTICAL WATER-POWER ENGINEERING

is indeed fortunate if he can secure reliable run-off records for a long and continuous series of years, or if he can find records of stream measurements from which he can obtain a reasonably good idea of the run-off. Of course, some countries are more fortunate than others in this respect.

Although a topographical survey may take many years to complete, when an area of country has been surveyed, to the degree of accuracy and precision contemplated, the survey of that area is complete. A hydrometric survey on the contrary is a progressive activity. It contemplates :—

1. The continuous gauging of all rivers and streams, not only when the rivers and streams are flowing at the normal and minimum discharges, but also at their flood discharge ; not at widely separated intervals, but at as frequent intervals as possible ; not at one point on the course of each, but at as many points as possible. The method adopted for attaining this end is the establishment of metering stations at the points selected on the courses of rivers and streams. In the case of all except the smallest streams, the form which a metering station usually takes is simply the installation of an erect graduated staff, firmly fixed at a suitable point close to the bank, or in a recess in the bank, the reduced level of the zero of graduation being a fixed and known figure. The cross-section of the river is taken at the site of the metering station. Observations on the velocity of the current in longitudinal sections of the river are made with a current meter at regular distances across the cross-section, and at either two or more depths at each point on the cross-section, some half-a-dozen times at various stages of the river. These observations enable the velocity and discharge curves for all stages to be plotted (see p. 83).

Subsequent observations at the metering station merely consist of the reading of the level of the water on the graduated staff at as frequent intervals as possible, a work which can be done by anyone who can read figures and decimals or fractions, and readings can be taken at a number of different metering stations, not too widely separated, in a day.

This method, though not so precise as gauging by weir, is inexpensive, and has been found in Canada and other countries to be sufficiently precise for practical purposes, for all except the smallest streams (see p. 73).

In the case of very small streams, it is advisable to install an inexpensive weir gauge in a permanent manner, and the level of the water passing over this is read on a small graduated staff in the same manner at frequent intervals (see p. 76).

2. The investigation and examination of the power reaches of rivers ; the production of maps and data showing the maximum

capacity of each power reach, for economic power transmutation, not by utilising the minimum flow of the river at the head available from the difference of level between the top and bottom of a pronounced waterfall, included within the limits of the power reach, but the maximum power the reach is capable of producing by making use of this flow, augmented by the increment derived from storage of flood at suitable sites, at the total head economically available from the difference of level between the upper and lower limits of the power reach.

3. The investigation of sites for reservoirs, suitable for the storage of flood waters (see p. 48).

4. The investigation of sub-surface supplies of water by trial bores.

5. The investigation of drainage and irrigation projects and sources of supply for other purposes, also observations on the evaporation from lakes, swamps, and soils, on absorption of soils and rocks, and discharge from catchment areas, in their relation to rainfall and humidity.

Before data can be collected sufficient to afford a reasonably accurate guide to operators, whether such operators be the Government, municipalities or companies or private persons, continuous gaugings extending over several years are necessary. The more extensive these gaugings are, the more reliable they are.

Evidently it is only after a long series of observations that the minimum, normal, and maximum discharges of rivers can be ascertained with accuracy. and at present certain Governments and companies desiring to operate are faced with much uncertainty regarding the continuous flow of a source of supply which can be relied on and the flood discharge to be guarded against.

Time is not always available to them, when a scheme is proposed, to make a continued series of gaugings, and in those cases where projects have been evolved and carried out on inadequate data, by Government, companies, and private persons, the operators have generally found to their cost, when too late, that they had relied on a much more constant flow than experience in operation proved to have been warranted. Cases are on record which show this to be only too true.

The Water Power Committee of the Conjoint Board of Scientific Societies, London, in their preliminary Report of July 1918, stated :—

“ 1. That : to be of reliable value from a commercial point of view, the hydrometric studies must give a continuous record for a number of years, and show not only the minimum low-water flow, but also the maximum flood conditions that have to be met in designing the head-works.

" 2 That : the investigation of suitable rivers should include contour plans of the sites, profiles along the entire power reach of the river, and along the banks ; also studies of lakes or lochs for storage, where they exist, and of the possibility of interconnecting two or more such lakes to feed one large project. These studies should be in sufficient detail to allow of preparing preliminary plans and estimating capital and operating costs, in order to demonstrate the capacity available and the commercial feasibility of development.

" 3. That : to develop the most obvious power site on a river without full investigation of the whole power reach of the river may not secure, and may make it impossible to secure, the maximum advantageous use of the river by the development of two or more sites.

" 4. That : to secure the maximum possible use of a river, the investigation should therefore be made by the Government rather than by private interests.

" 5. Especially is this the case where storage may be developed, in order that the maximum possible storage may be secured, and that the water may be equitably distributed to, and the cost of the works equally borne by, the various interests benefited. Proper storage may greatly improve flood conditions and enhance the value of land as well as increase the power available.

" 6. That : without complete surveys the complete capacity of a river cannot be accurately judged. The pondage created by the dam will in many cases more than take care of the daily peak load, thus increasing the power available beyond that due to the minimum low-water flow, and this may be still further increased by storage at the head-waters. The power capacity of a river may sometimes be increased by such means by 100 or 150 per cent. or more."

If no records are available, the first preliminary step is to measure the cross-section and slope of the river and to calculate the probable run-off and compare the catchment area with adjacent catchment areas, as also the known run-off (if any) by means of the rainfall records if the latter can be relied upon. In cases where reliable records are not available, and the water-power project appears to be a satisfactory one, it is then advisable to establish measuring stations (where possible to obtain at relatively little expense) and/or put up rain gauges at the best localities in the catchment area. This will often mean the construction of weirs for measuring the stream flow. It will invariably mean the establishment of current meter stations. Ultimately, no matter what the method of calculation may be, the object in view is the determination of the number of cubic feet of water per second which will be available for each day, month,

and year. In any case there will always be the problem of how closely the record for a given term of years will represent the flow to be expected during the life of the development.

The run-off from any catchment area fluctuates between very small quantities at times of drought and quantities sometimes many hundred or even many thousand times with, usually, little regularity about the occurrence of the extreme or intermediate stages of stream flow. Moreover, the average usual discharge for some years may be considerably lower than for others, hence the desirability of knowing precisely the lowest ordinary minimum flow of the river.

Rarely is it commercially practicable to develop a water-power plant for an output based on *the extreme minimum* stream flow. Without storage or auxiliary steam or other power plant, the maximum power which can be developed continuously nearly always depends on the water available during the period of *ordinary minimum* flow. The ordinary minimum flow referred to here is the mean of the average flows during about two of the lowest consecutive seven-day periods during the period of record (as decided upon), which period of record should extend over and be complete for a term of years (seven or more, as found best). In general, the extreme minimum stream flow lasts for only a very small portion of the year. Nevertheless, these records are of great value. In the determination of the maximum practicable economical development, several different methods are in use, but whatever method is used, much will depend on local conditions and circumstances and other considerations. With storage, the most economical development can be determined by obtaining the available power from storage, and balancing its economic value against the charges to be debited against the storage, etc. Due to circumstances, etc., which cannot very well be changed, over-development of water power is *the general rule*. The total cost of hydraulic and electrical plant in most cases represents only a small percentage of the total cost of the complete hydraulic works, which works must be completed for two main reasons, namely: (a) to provide, at the least cost and in the quickest time, a maximum supply of power to meet every growing and immediate demand, and (b) to avoid prohibitive costs in carrying out extension of the hydraulic works where the latter are completed only for part of the total water-power capacity of the river. In general, it is almost as expensive to develop a small amount of power as to develop the stream to its full development capacity. As the cost of a dam is proportional to its length and height, the cost will not vary materially with the capacity of the power plant. It may so happen (for a given river) that during the flood stage a water-power plant, *if not properly located*, may be completely incapacitated through loss of head by back-water.

On the other hand, the water-power plant may be so located with reference to tail-water level (due to an oversight of or ignorance of the true value of extreme minimum flow) that the practical working draft-head has been exceeded.

In establishing the relationship between rainfall and run-off, factors such as geology, topography, size of catchment area, vegetation, distribution and intensity of the rainfall, temperature, etc., must all be given fair consideration. Moreover, the ratio of the total run-off to rainfall will depend largely on the evaporation, and the latter will depend on such factors as velocity and humidity of prevailing winds, exposure and temperature, state and amount of vegetable growth, etc. It is obvious that a great deal must be left to the judgment of the investigating engineer not only in the matter of the state of accuracy and completeness of the studies, but also in the application of a reliable run-off coefficient. A very desirable condition (for the engineer) is to have thoroughly reliable information available from actual discharge measurements taken over long and complete periods of time, together with comprehensive information of the catchment area, precipitation, etc., entering into the particular case. A safe maxim is to deal always with stream gauging in preference to collection of rainfall data.

In the technical working out of a water-power project it is best to follow along the broadest lines since the objective is to indicate and fix upon a definite scheme which will utilise to the maximum extent the *full power resources* of the river. The full scope of such work is usually very comprehensive. It may not only involve a close study of the complete water-power project requiring reliable information of all features and aspects of the scheme, but it often requires a detailed investigation and study of the load factor, diversity factor, the general conditions and character of power, lighting, and other load anticipated, as well as a determination and safe estimate of the cost of the proposed development, etc. There is also the legal aspect involving not only the possession of the water rights, but also right of lands, right of ways, etc. Also, before a report is favourably received, it is necessary to know whether the vested interests and other properties, options, etc., are legally sound. The people offering to finance the project would invariably engage a legal man to scrutinise carefully all legal matters and advise them accordingly.

In selecting the best and most economical sites, the engineer may have to compare one site with another, and get together and put into proper shape information of an intricate nature, involving such questions as: information giving fully the scope of investigations; the course and approximate length of river under investigation; route of river; gradient of river; location of gauging stations

(giving the two extreme levels); duration of floods; condition of river bottom and banks; effects of sharp bends on flood flow, etc.; inundated lands, giving their extent and cost; territory surveyed; catchment area or areas; principal rivers and tributaries; rainfall records of districts, etc.; impounding flood water; run-off water; average daily or monthly run-off for development of power; natural reservoirs; artificial reservoirs; location of dam, of head-works, conduit, pipe line, etc.; areas and altitudes; geological features; mineral resources in the territory; road diversions, etc.; vested interests; landowners; navigation and fisheries; towns, villages, population; transport facilities (giving access to proposed sites); route; foundations for works; scheme recommended; financial effect of impounding water, etc., etc.

A careful reconnaissance of the head-waters of a river would be necessary to provide the preliminary information as to possible good sites for storage, together with the nature of foundation material of the reservoir-bed and the capacity of the reservoir; the probable flow at the proposed site of dam; materials available on or near the site for its construction, and all other information that might have a bearing on the practicability and economic aspect of the site not requiring too much time and expense to ascertain. If no suitable reservoir site can be located on the river itself, an examination of the surrounding country must be made to find whether there is a suitable site on which a conduit of economical length could be constructed from the river. Examinations may also have to be made of the adjacent catchment areas and rivers. The dividing ridge or ridges may also have to be examined to ascertain if it would be economically possible to divert water from one watershed to another, assuming that the Government concerned (Colonial, Home, or Foreign) would allow such a step to be taken. These examinations, if carried out in detail, are usually quite expensive—for most preliminary purposes, rough examinations will ordinarily suffice.

An examination of the catchment area will determine the feasibility of storage reservoirs to hold back floods so that the water may be used during low-water months of the year. The principal objective in this case would be one large area or several areas (preferably the former) for impounding flood waters; the sites offering good foundation material for a small dam to hold them.

The survey of a natural lake may include an investigation of the possibility of storage by damming the lake outlet in some way or other, or a diversion of the water to another watershed by tunnelling, etc. The boundary of the lake may be extensive, and profiles may have to be run up the slopes at 5-ft. contour intervals to a height sufficient to cover the highest elevation to which the water may be

raised. Test pits or borings may have to be made at the selected site to give a good general indication of the nature of the bed of the lake where it is proposed to build the dam. The best type of dam for a particular location will depend to a great extent upon the nature of its foundations, the profile of the dam site, the materials available for construction of the dam and perhaps other hydraulic works, accessibility of the site as regards transportation facilities, and other considerations, etc. A poor foundation may prove fatal to the success of a water-power project. Such a dam may be required to (1) form the complete head-works at the intake of a conduit; (2) create storage; (3) create a working head; or (4) increase the working head. Want of proper building sand and materials in the vicinity may cripple the development entirely or greatly increase the cost of it. Good rock foundations and abutments are always favourable assets. While carrying out these examinations, careful records should be made of rock outcrops, giving their classification, also records of all other classes of material such as gravel, sand, and loam, etc., which may be in evidence. With regard to accessibility of sites, notes should be made of the nearest railway line and/or siding, giving distance, etc.; approximate cost of constructing a branch line to the site, the condition of rock on or near the site or the railway route and its suitability for heavy traffic, etc. In taking care of the water during the construction period, the absolute necessity and cost of a cofferdam should not be overlooked. The physical conditions at the site of a dam and the appurtenances are about the most important factors affecting the cost of construction.

With regard to the selection of point of diversion from the river to the conduit at the head-works site, a preliminary examination should indicate in a general way the elevation at which it is necessary to divert the flow for power purposes. A close examination should be made for the purpose of locating the most suitable and the most economical diversion for the head-works and conduit intake. Generally, it is found necessary to build a dam or weir across the river at the point of diversion. This may mean that a certain amount of land will become flooded and, although relatively small in area, it may be very valuable and difficult to acquire. Before finally deciding on the diversion site, an examination should also be made of the banks of the river and its foundation material at the site of diversion, as these decide in a general way such matters as the type of dam, its height and length, etc.

In the matter of conduit location and the *complete* carrying out of the conduit survey, the same engineer carrying out the above-mentioned work should be employed or, failing this, an experienced topographical surveyor should be employed. Unlike several other

main links in the hydro-electric "chain," a conduit cannot very well be built in duplicate, and once a conduit is built it can only be taken down and relocated or replaced at prohibitive cost. Very often this part of the hydro-electric system represents a large percentage of the total cost of the complete hydro-electric development. In deciding on the kind of structure to adopt for the conduit, *i.e.*, wooden flume, concrete or masonry section, etc., the first cost may be, and sometimes is, relatively less important than the losses, cost of operation and maintenance. As regards the conduit location, this is usually a simple matter, but to locate a conduit *economically* is an entirely different question, requiring broad experience in this class of work. As regards design, the most common problem in ordinary practice is that of determining the dimensions of the conduit for a given discharge and a given slope of the conduit bottom, the latter being the variable factor (see p. 106).

In connection with the question of water-power sites it is sometimes found necessary to locate a steam plant or several steam plants to operate in proper relation with the hydro-electric system, including the power transmission lines, etc. Such a steam plant (or plants, as the case may be) may be required as a reserve in case of interruptions to the hydro-electric system, or it may be used to generate a substantial portion of the energy output or to generate energy to carry some of the system load over a period when the water-power plant or plants are loaded and before the demand increases to a point where the large initial cost of a modern water-power plant is justified, or it may be required to generate energy in years of extreme low water, or to carry a fair portion of the peak load, these two latter usually being its principal function.

No matter how close or how widely separated water-power sites may be, it rarely occurs that the natural disposition of two sites are identical in character. Practical experience with one set of conditions may be of little practical help in the study and selection of water-power sites offering quite different conditions, etc. The power site shown in Fig. 8 is that of a water-power development now operating. According to the plan and profile, the project shows quite a satisfactory set of natural conditions. It is seen that part of the river (above the diversion site) is used as a conduit to the system of waterways, and that the water supply is regulated at the impounding-dam. It shows that the impounding-dam and the diversion-weir are of but moderate length. It shows a very short gravity-conduit and a fairly large pond for the forebay. Also, the hillside for the pipe lines has quite a satisfactory gradient, etc. In a comparatively short distance a head of 350 ft. is secured; without impounding the water at A a continuous flow of 800 cusecs is obtained. This head and

flow represents a gross water power of 31,750 h.p. Such satisfactory results as given here cannot be said of a vast number of water-power projects and sites already developed and contemplating development. This particular site is mentioned because it incorporates nearly all the elements involved in hydro-electric work, *i.e.*, dam, river-conduit, weir, gravity-conduit, forebay, pipe line, and power house, etc. The principal reasons for the choice of this power-reach are obvious from Fig. 8 (see p. 63). However, let us look into the question of available power from this reach.

The development consists of a main dam, a diversion-weir, a conduit, a forebay, four pipe lines, a power house, and a tail-race. The working head is 336 ft., and the estimated constant (twenty-four hour) flow, as obtained from reliable stream flow records, is 800 cusecs; it is estimated that this rate of flow can be very much increased by proper use of the water impounded in A and by a close study of the load factor and peak loads.

The main dam A is built at the outlet of a lake, which latter is slightly over 10 sq. miles in area, and it provides for a storage capacity of $9 \times 640 = 5,760$ acre-ft., 1 ft. in depth, with a maximum draw-down of 23 ft., making a total capacity in acre-ft. of $5,760 \times 23 = 132,000$ acre-ft.; say, 132,000 acre-ft. after allowing for losses and the larger area, etc. In theoretical units this would be equal to

$$\begin{aligned} 132,000 \times 1.025 &= 135,000 \text{ kW.-hours} \\ 132,000 \times 1.374 &= 181,500 \text{ h.p.-hours} \end{aligned}$$

per *foot-head* at 100 per cent. hydraulic efficiency.

For this plant it was estimated that the average continuous (8,760 hours) yearly rate of flow could be safely taken at 800 cusecs. Taking this as the twenty-four hour rate of flow in cusecs, then the equivalent average twenty-four hour rate of flow in cusecs which the storage is called upon to yield with no flow into it, is 69.12 million cub. ft., that is,

$$P_d = P/86,400 = 69,120,000/86,400 = 800 \text{ cusecs or ft.}^3/\text{sec.},$$

then the twenty-four hour rate of flow in cusecs which is completely equalised by storage would be the same for 100 per cent. load factor, but not so for a lower percentage load factor.

The diversion-weir (like the main dam) is built of concrete, and is partly reinforced at the intake *a*. The design of spillway and gates of the main dam A provides for a flood discharge capacity of 10,000 cusecs, which is estimated to be in excess of the highest recorded outflow from the catchment. At the intake *a* it was estimated that a continuous flow of 800 cusecs could be maintained for use in the development of water power from the river flow. The conduit was designed and constructed to handle a greater rate of flow than

800 cusecs. For the rate of flow given, the available power per foot-head per second at 100 per cent. efficiency is

$$\begin{aligned} 800 \times 0.1134 &= 90.8 \text{ h.p.} \\ 800 \times 0.0846 &= 67.8 \text{ kW.}, \end{aligned}$$

that is to say, 90.8 h.p. under 1 ft.-head at 100 per cent. efficiency is equivalent to $8.82 \times 90.8 = 800$ cusecs.

Hence, at the intake *a* we have a power representing approximately

$$90.8 \text{ h.p.} = 67.75 \text{ kW. per ft.-head.}$$

The conduit *ab* will permit a much greater flow than 800 cusecs. It is built of concrete section; the coefficient of roughness was taken at 0.013. Between the intake-conduit and forebay the gradient averages 0.0022 ft. per foot or 2.2 ft. per 1,000 ft. of conduit length. The total difference in head between *a* and *b* is 7.5 ft.; thus, taking efficiency of water from the conduit-intake to the forebay at 98 per cent., including losses to supply pipe entrance, we have

$$\frac{7.5 \times 800 \times 62.4 \times 0.98}{550} = 667 \text{ h.p.},$$

or power loss due to loss of head and water is

$$7.5 \times 800 \times 0.1119 = 667 \text{ h.p.}$$

Topographical and geological conditions at the forebay site B are such that a concrete structure became necessary; also, the area of the forebay was somewhat limited by excessive cost of the construction, but fairly good pondage for regulation is provided. The forebay is built in two sections: (*a*) one section into which the water discharges direct from the conduit, and designed to form a settling basin with flow sloping to a sluice gate; (*b*) a section fitted with two sets of strainers (racks), forming the intake-chamber and entrance to the pressure pipe lines.

The pressure line consists of four pipe lines, each averaging 72 in. diameter and $\frac{1}{8}$ in. in thickness of tube (precisely each pipe line consists of 96-in., 72-in., and 50-in. diameter pipe, with $\frac{1}{8}$ in. thickness at the top section and $\frac{9}{16}$ in. thickness of shell at the lower section). The pipe is double and triple riveted (lap-joint) steel pipe; the types of joints have efficiency of 73 and 75 per cent. respectively. The total length of each pipe line from the forebay to the power house is about 13,700 ft. The surveyed mean fall from the water surface in the forebay to the tail-water level is 350 ft.; the gradient of the pipe lines averages 25 ft. per 1,000 ft. Deducting head loss in all the pipe lines (which was computed to be 14 ft.), the estimated net head at the turbine is 336 ft. This would therefore represent a power loss in the pipe lines of

$$14 \times 800 \times 0.1134 = 1,270 \text{ h.p. at 100 per cent. water efficiency,}$$

or

$$14 \times 800 \times 0.1100 = 1,230 \text{ h.p. at 97 per cent. efficiency.}$$

For this project the engineers assumed (for the given net flow and head) an *over-all* efficiency at the turbine-shaft of 80 per cent.; the over-all efficiency as referred to here meaning the sum of the efficiencies of all the various parts from and including the diversion site all the way to the turbine-shaft. The total brake-horse-power output, therefore, represents

$$336 \times 800 \times 0.09076 = 24,390 \text{ b.h.p.,}$$

or

$$336 \times (800/0.80) \times 0.1134 = 24,390 \text{ b.h.p.,}$$

the power house provided for a turbine capacity of four 5,750 h.p. units with space for additional units when required.

In computing power and energy it is first necessary to estimate the run-off at the given site for the type of year most pertinent to the particular conditions. The most probable monthly or yearly load factor is very important where storage is used. It is very necessary to collect and analyse all the base data, such as the available rate of flow, net head, storage, load factor, etc. With reference to the load factor, it would be better to compute for a range of yearly load factors from about 30 to 70 per cent. Also, commencing with the minimum flow, calculations should take the form of percentage of time values; preferably by 5 or 10 per cent. intervals. For these different intervals of percentage of time the power is available, a curve should be plotted for values ranging from 100 down to 50 per cent. The average rate of flow and the available power and energy should also be plotted on the same diagram.

In this project the lake A is purely for regulating purposes, whereas the forebay B is used for taking care of the fluctuations of load and sudden demands for power. The *possible* storage area of the lake is about $9 \times 27.88 = 250.92$ million sq. ft., or 250.92 million cub. ft. of water 1 ft. deep. This area of water is equal to

$$250.92/0.0864 = 2905.3 \text{ cusecs for one day of twenty-four hours.}$$

As explained above, it was considered safe, when framing the base data, to make the plant capacity 24,390 b.h.p. with a net available head of 336 ft. This is, of course, based on a 100 per cent. load factor; however, the available power in the river is materially changed for a lower load factor. Thus, for 800 cusecs flow, the reservoir A will completely equalise all daily flows at 35 per cent. load factor up to 1,230 cusecs; that is, let

$$Q_2 = Q_1/(1 - Y); \text{ then: } 800/(1.0 - 0.35) = 1,230 \text{ cusecs.}$$

When the average twenty-four hour rate of flow is less than the rate

of flow which is completely equalised by storage, then the average of flow available during the working hours is expressed

$$Q = Q_h/Y = 800/0.35 = 2,285 \text{ cusecs.}$$

Above this rate of flow, the average rate of flow available during the working hours will be

$$Q' = Q_h + 2,285 = 800 + 2,285 = 3,085 \text{ cusecs.}$$

The average rate of flow during the working hours supplied from storage will be

$$Q_p' = Q_h(1 - Y)/Y = 800 \times (1.0 - 0.35)/0.35 = 800 \times 1,857 = 1,486 \text{ cusecs.}$$

But, when the average rate of flow (twenty-four hour rate) is greater than the twenty-four hour rate of flow which is completely equalised by storage, then the average of flow available during the working hours is

$$Q' = Q_h + Q_p' = 800 + 1,486 = 2,285 \text{ cusecs.}$$

The available power for twenty-four hour rate of flow (assuming an over-all efficiency of 80 per cent., which allows for total leakage and loss of water and turbine efficiency) is

$$\begin{aligned} \text{h.p.} &= (Q_h H)/11.0 = 0.090768 Q_h H \\ &= (336 \times 0.090768) Q_h = 30.5 Q_h \\ &= 30.5 \times 800 = 24,400 \text{ h.p.,} \end{aligned}$$

but the available power at 35 per cent. load factor is

$$30.5 \times 2,285 = 69,690 \text{ h.p.,}$$

and the total available energy for 100 per cent. load factor is

$$30.5 \text{ TY} Q_h = 30.5 \times 8,760 \times 1.0 \times 800 = 213,744,000 \text{ h.p.-hours.}$$

The load factor of a station has nothing to do with the quantity of plant installed. It is a measure of the constancy of the demand for power throughout the year; that is, it is the proportion between the number of horse-power hours actually generated in a year and the number which would have been generated had the maximum load been carried during the whole period. The total cost of generating power is taken as the sum of the capital charges on the plant and the expense of operating it. So far as the capital charges are concerned, the effect of the load factor on the cost of power is a matter of calculation; that is to say, if a station has cost £30 per h.p. of its maximum load, and if the load factor is 20 per cent., every horse-power hour sold will have to bear the burden of about 0.3d. over and above the operating costs of the station. Increasing the load factor to 40 per cent. might reduce the capital charges to 0.15d. (pence) per h.p. hour.

In the United States of America only such projects and sites are approved by the Commission as are in its judgment best adapted to a comprehensive scheme of improvement and utilisation for the purposes of irrigation, of water-power development, and of other beneficial uses. The factors to be decided must show that :—

(a) Full practicable utilisation will be made of the water-storage possibilities and the head at the site to be developed.

(b) The structures must be in accordance with good engineering practice.

(c) All unnecessary energy losses, whether in hydraulic works or in mechanical or electrical equipment, must be avoided.

And that :—

1. Whether in relation to existing or probable future projects upon the same or adjacent streams, the fullest practicable utilisation of the water-storage possibilities and head available must be made possible.

2. Whether the project will be in general accord with the most beneficial utilisation of the water for navigation, water power, irrigation, or other beneficial public uses, and for aiding flood control, reclamation, and similar developments.

3. Whether proper provision is made for present or future electrical interconnection with other projects or systems in order to take advantage of diversity of stream flow and of power demands.

4. Whether the use to which the power will be devoted is, in general, in accord with the needs of the community and of the public welfare.

5. Whether the applicant is financially able to carry out the development.

6. Whether the construction, maintenance, and operation of project works will interfere or be inconsistent with the purpose for which any reservation was created or acquired.

The power capacity of a project usually means that amount of power estimated to be available for transmission, and is determined as the continuous product of (a) the average or the effective static head in feet ; (b) the available stream flow at the intake, in cusecs and not in excess of the hydraulic capacity of the approved project works, estimated to be available from natural flow or from storage, or from both ; (c) a factor, not less than the average load factor of the power system, representing the degree of practicable utilisation of the available stream flow, etc., and based on the extent of practicable forebay and/or storage and the load factor of the power system ; (d) a factor representing the horse-power at 70 per cent. efficiency of one cusec falling through a head of 1 ft., say 0.08. The load factor as defined here meaning the ratio of average power output

to maximum power input. The available stream flow may mean the sum of nominal stream flow made practicable by the project works ; nominal stream flow meaning the sum of (1) the average of the values estimated for the mean natural flow for the two months' (calendar) minimum-flow period in each successive five, four, three, or two year cycle ; and (2) the increase in such average due to artificial means other than the project works.

Apart from the many factors (fixed, variable, and uncertain) involved in the proper selection of sites, the engineer selecting water-power sites must also keep well in mind possible hydraulic losses, such as those due to leakage, seepage, evaporation, roughness factor (coefficient of roughness), tail-water level and back-water conditions, general losses in all the developed waterways such as those occurring in the conduit, pipe line, and turbines, etc. Commencing with precipitation, and taking in the catchment area, including its complexity of conditions, storage, stream flow, and methods of harnessing the stream, etc., a long list of losses are in evidence. In the undeveloped waterways and those areas affected by precipitation, varying degrees of conservation can be effected in many ways. In the developed waterways themselves, *i.e.*, conduit, pipe line, storage or pondage, turbines and tail-race, improvements can be made in the way of more careful estimating of the stream flow ; reduction in leakage and overflow, reduction in intake losses due to eddies, whirls, etc. ; more efficient flashboard work ; lining of open and closed conduits ; close survey work to reduce losses due to grade ; better maintenance of a higher forebay level ; proper cleaning of trash racks ; elimination of obstructions in all the waterways ; operation of turbines at proper speeds for maximum power and efficiency ; proper turbine setting ; reduced leakage at the runner seals, etc. Finally, in the construction of the hydro-electric works a point to bear in mind is that successful operation requires quick construction and quick use, and that the incurred interest up to the time the project is earning full income soon becomes a large part of the fixed charges (see Chapter VIII.).

In many instances power stations can only be economically placed on sites which at intervals may be subjected to flood conditions, and while available sources of information with regard to maximum water levels indicate the safety of the situation, an excessive flood, and, in some cases, an obstruction due to the position of part of the hydraulic work, may affect the normal characteristics of the river flow sufficiently to cause high tail-water or failure. Failure of hydraulic equipment or excessive rise of tail-water may cause powerhouse flooding. One of the most difficult problems to solve with exactness is the height of back-water, caused by a dam under

the different conditions of stream flow and the resulting limits of flowage. The failure of a section of conduit may completely destroy the hillside, and the failure of a forebay or pipe line may destroy both the hillside and the power house below. Failure of the project, taken as a whole, is usually due to such matters as over-estimation of the quantity of water available; under-estimation of the cost of the development; over-estimation of the market for power; and under-estimation of competition in the sale of energy.

Useful Information

Good Foundations for Dams.—Dolerites or other igneous types, and gneisses and schists, quartzites, hard limestones, and even fine textured sandstones, if free from open joints and fissures, make good foundations for dams.

BEARING WEIGHT OF FOUNDATIONS OF VARIOUS ROCK AND EARTH

Material.	Tons per Square Foot.
Granite masonry or basalt — — — — —	25 to 30
Hard limestone — — — — —	25
Shales and hard sandstones — — — — —	15 to 20
Conglomerate — — — — —	12
Soft sandstone — — — — —	10
Hard shale — — — — —	8
Fine, coarse sand or gravel — — — — —	5
Rammed, dry clay, and firm, coarse sand — — — — —	4
Dry sand and hard, dry clay, blue clay — — — — —	3
Ordinary clay or dry mixed sand with clay — — — — —	2
Soft clay and loam — — — — —	1

WEIGHT OF ROCK AND EARTH IN LB./FT.³

Material.	Lbs. per Cubic Foot.
Slate — — — — —	157 to 180
Granite — — — — —	163 „ 170
Limestones — — — — —	160
Sandstones — — — — —	150 to 160
Chalk — — — — —	145
Cement and cyclopean masonry — — — — —	140
Clay, damp and wet — — — — —	120 to 135
Very wet sand — — — — —	125
Very wet earth — — — — —	115
Moist sand and gravel (spare) — — — — —	110
Dry sand and round gravel — — — — —	100
Moist earth — — — — —	95
Dry earth — — — — —	90

Note.—A well-constructed masonry dam should have a density of from 2.2 to 2.6; or, $62.4 \times 2.2 = 137.3$ lb./ft.³ to $62.4 \times 2.6 = 162.3$ lb./ft.³.

CHAPTER IV

STORAGE AND PONDAGE

ONE may have in mind that general matters in hydrology, such as rainfall, run-off, stream flow, storage, and other considerations affecting the hydraulic end of the hydro-electric system should not directly concern the electrical engineer. The writer takes the opposite view, based on experience, and emphasises the point that there should be no question of doubt about this matter, as, in most countries where hydro-electric power is extensively used, adequate experience has already made it definitely understood that these matters do, in general, concern the electrical engineer as much if not more than other branches of hydro-electric practice. For a moment let us look at the matter in a different way. Usually it is accepted that the electrical engineer is as concerned with the whole steam end of the steam-electric system—including general knowledge of the calorific value of, transportation of and delivery of fuel, as well as other considerations affecting steam generation and losses, etc.—as any other branch of steam-electric practice, and that his duties necessarily involve a thorough study and investigation of every item affecting efficient generation and utilisation of steam heat and steam power. Just so with this other branch of engineering, the duties of the electrical engineer might commence at the watershed (catchment area), and, just as he or his company (as the case may be) buys the temporary services of a *purely* civil engineer to design and build the dam, hydraulic works, buildings, and similar structures, in like manner the services of a hydraulic or mechanical engineer are obtained to put in the turbines and penstocks, etc., exactly as in the case of the steam-electric system for which a *purely* mechanical engineer is temporarily engaged to design, manufacture, and erect the boilers, auxiliaries, steam turbines, etc. Thus, as in the case of a steam-electric system, the entire hydro-electric system is invariably operated by an electrical engineer trained for such duties and experienced in such matters. The short-sighted policy of forming “water-tight compartments,” which was usually to stop outside of the power house or at the head-works, has practically died out. At the present time we have many hydro-electrical engineers, and the near future is sure to add greatly to their responsibilities and number.

As regards the reservoir itself, its three functions are : (1) inflow or filling ; (2) holding ; and (3) outflow or using of the supply of water as required.

The main features determining the amount of storage are : (a) dependability of water supply ; (b) the topography, with reference to angle of repose of the ground, etc. ; (c) the geology, with reference to character of bed and banks of the reservoir ; (d) the effective depth of reservoir ; (e) the locality with respect to the development as a whole ; (f) the character of, and cost of the dam or wall ; and (g) the cost of inundated land. In working up a study of storage possibilities, it is advisable to cover the whole subject as thoroughly as conditions may require before commencing the development, because inadequate study and investigation may result in complete failure of the hydraulic works and system. Having regard to the great cost of impounding water, it is of the greatest importance that adequate information should exist for estimating (more particularly in the case of the high-head development) the *rate* at which the available precipitation is yielded, and for determining therefrom the best available or most economical storage capacity and its costs.

In making a reservoir survey, contours of from 4- to 10-ft. intervals are common practice, particularly the former, or 5-ft. contours ; very large storage areas have verticals taken to 10 ft. A survey is considered to be good if accurate within the range of 8 to 16 per cent., depending on the reservoir area and its topography. If the profile is plotted to large scale, then all rock-outcrops, as also water levels and estimated height of back-water, should preferably be shown on the contour map. It is important that contours be extended above the highest elevation to which there is any possibility of taking the head-water. In the preliminary survey enough instrument work should be done to permit of plotting a fairly accurate contour plan of the whole area covered by the proposed scheme. If not too complicated, water elevations for extreme flood and low flow should be plotted on the contour plan or map, and back-water levels should be marked in. A profile should also include section through the dam, intake, etc., and should show the governing features of elevation, such as water-head, crest of dam, and other necessary information. The height to which the dam should be built may be limited by the total cost of flooded land, or perhaps the topography above the site of dam may control the height. It will also depend upon the amount of foundation clearance, amount of freeboard, and full supply and high-flood levels. To ascertain the slope or fall of the stream to the proposed site of dam, elevations of water level should be read and plotted ; these will indicate the approximate location of the dam and head-works. For storage capacity, a slope less than 10 ft. per

mile is considered good. Special attention should be given to the reservoir outlet, as conditions of foundation material for dam, character of banks, height and slope of banks, section across the stream, etc., may affect the type of and cost of dam, spillway, and complete construction. A good dam site can often be found below the junction of two streams (see p. 38).

Storage dams vary from a long and comparatively low wall to a short and very high structure. These latter structures call for the services of an expert in dam construction, and is a part of the hydro-electric development rarely if ever left to the regular hydro-electric engineering staff. Another type of dam (more usually spoken of as a "weir") is placed at the intake of the developed waterways; the spillway crest of this type of dam is made lower than that of the dam or the surplus waterway pass. The former type of dam is used for water-storage purposes, whilst the latter type of dam is commonly used for diverting the natural stream into the developed waterway (conduit, etc.). Depending on the type of development, flowage lands which have been purchased and only used during floods may be utilised to increase the operating head and pondage by the use of gates in the dam instead of an open spillway. Such gates might be used for keeping the head-waters at a maximum height during low water and provide high discharge capacity at times of flood (see p. 66).

The height to which a dam should be built is sometimes limited by the total cost of inundated land. The natural stream above and below the dam (not the dam itself) may also control the height to which the water rises; this is sometimes found to be the case where the dam is built at the head of rapids or in the bend of a river. The location, arrangement, and cost of the head-works is sometimes governed almost entirely by the natural conditions. Ordinarily, low-head plants simply have a dam, whilst others, in addition to the dam, require a conduit and forebay, the aim being in every case to secure the greatest amount of energy with the best all-round structural advantages and least cost; in other words, to obtain the highest head possible—for a given flow—at the least cost. The best location and arrangement of the forebay is usually a matter of cost, but with favourable topographic and geologic conditions the most effective pondage for regulation of flow can be made at relatively low cost; its location and arrangement must be such as to deflect all foreign matter from the entrance to the pressure pipe line (penstock).

Masonry dams are, for the most part, merely retaining walls of exceptional size, in which the overturning pressure is water. In this text it is not intended to treat with design of dams, and it is hoped that a mention of a few basic principles will not be out of place.

As the pressure of water is zero at the surface and increases in

direct proportion to the depth, the overturning moment is as the cube of the depth; the form of stability having the least sectional area is a triangle. For masonry dams having a density of *three*, water being *one*, it can be shown that the figure of minimum section is a right-angle-triangle, with the water against its vertical surface. For a given density, the water face must lean towards the water; for a less density, away from the water. For the sections of masonry dams actually used in practice, if designed on the condition that the centre of all vertical pressures when the reservoir is full shall be at two-thirds the width of the base from the inner toe, the least sectional area for a density of two also has a vertical water face. As the density of the heaviest rock is only three, that of a masonry dam must be below three, and in practice such work if well constructed varies from 2.2 to 2.6. That is to say, if a right-angle-triangle, Fig. 4, *aba*, be a profile of 1 ft. thick of a monolithic dam, subject to the pressure of water against its vertical side to the full depth $ab = d$ in feet, the horizontal pressure of water against the section of the dam, increasing uniformly with the depth, is represented by the isosceles right-angle-triangle *abe*, in which *be* is the maximum water pressure due to the full depth d , while the area $abe = d^2/2$, is the total horizontal pressure against the dam (generally stated in cubic feet of water) acting at one-third ($\frac{1}{3}$) its depth above the base. Hence, $d^2/2$ is the resultant horizontal pressure with an overturning moment of $d^3/6$. Now, if y be the width of the base, and p the density of masonry, the weight of masonry in terms of cubic feet of water will be $(pyd)/2$ acting at its centre of gravity g , situated at $(2/3)y$ of the outer toe, and the moment of resistance to overturning on the outer toe $(py^2d)/2$. Equating the moment of resistance to the overturning moment, we have

$$\frac{py^2d}{3} = \frac{d^3}{6} \quad \text{and} \quad y = \frac{d}{\sqrt{2p}}.$$

That is to say, for such a monolith to be on the point of overturning under the horizontal pressure due to the full depth of water, its base must be equal to the depth divided by the square root of twice the density. For a density of 2.5 the base must therefore be 44.7 per cent. of the height (see p. 46).

In practice, certain assumptions are involved in which the moment of resistance depends upon the figure and the weight of the masonry. One such assumption is that, if the dam is well built, the intensity of the vertical pressure will vary nearly uniformly from face to face along any horizontal plane. If we assume the water to have a depth d above the base, the total water pressure represented by the triangle *h'bh* will have its centre at $d/3$ from the base, and assuming the density

of the masonry to be 2.5, we find that the centre of pressure upon the base bc is shifted from the centre of the base to a point x' nearer to the outer toe c , and the rectangular diagram of pressures will thus be distorted from the figure $bf'g'c$ to the figure of equal area $bf'g''c$,

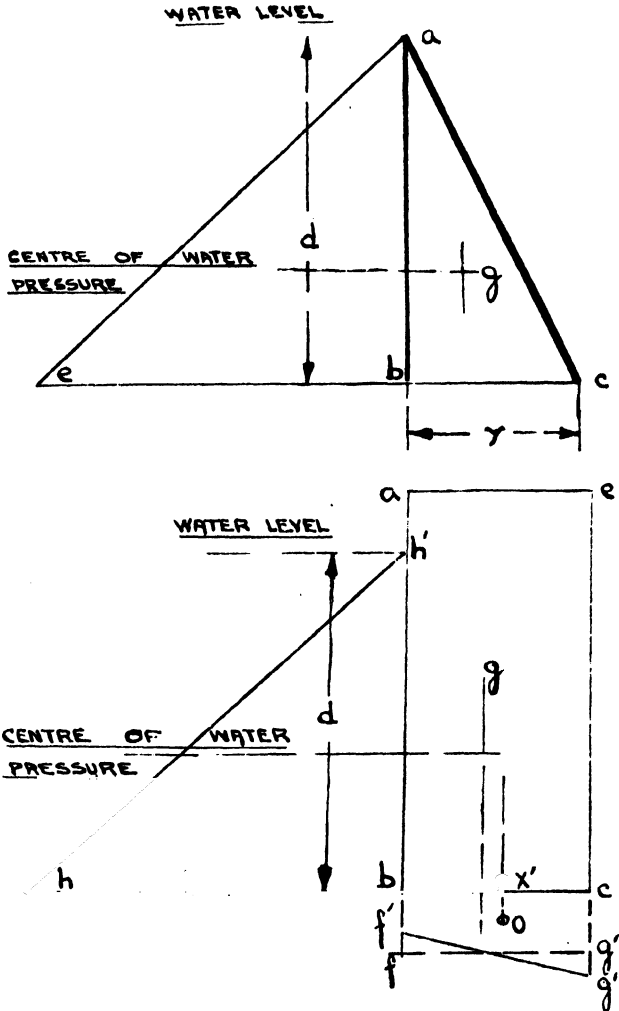


FIG. 4.

having its centre O vertically under the point at which the resultant of all the forces cuts the base bc . For any lower level the same treatment may (step by step) be adopted, until the maximum intensity of pressure exceeds the assumed permissible maximum, or the centre of pressure reaches an assigned distance from the outer toe c , when

the base must be widened until the maximum intensity of pressure or the centre of pressure is brought within the prescribed limit. There are many excellent text-books on this subject which should be referred to when dealing with dam design and construction; these few brief notes refer to the principal fundamentals.

Apart from the several points mentioned (see p. 38), the success of a dam will depend on: (a) suitability of the foundation; (b) capacity for maximum flood; (c) gates to keep low-water head at maximum while providing ample capacity—with gates open—to keep head and back-waters at their designed limits; (d) strength and stability; (e) how tight the ends and base are against seepage; (f) the provision against erosion downstream from the structure; (g) the provision for fishways, valves, or gates; (h) whether it is *long* enough for passing floods and *short* enough to avoid high cost.

The power-producing possibilities of a catchment are, for convenience, sometimes illustrated in the form of hydrographs, as shown in Fig. 5. Such a hydrograph is sometimes constructed to show the amount of water which is flowing past the site projected. This is usually extended over as long a period as there are stream flow and other records of run-off, and the minimum flow chosen which is to form the basis of design for the development of power. If, however, the development is to contain storage, it is then possible to regulate the flow, and in the complete investigation a mass curve of the river discharge is constructed for the average run-off for a period of years. In carrying out such an investigation, the available power from the regulated flow can be determined for previous years for which records are at hand. It may be represented by a curve of power plotted to calendar time, or it may be represented by a time-flow curve which indicates for what percentage of the time-period any given amount of power is actually available. This latter curve is the most important, as it shows the minimum power and the percentage of time during which power equal to or greater than any given amount was available, also its amount for any given percentage of time. From the mass curve of run-off, the available water for power is obtained, and this, under given effective head, will determine the amount of power available—this latter being the information required; but, before it can be given, we must first know the available run-off values. A point to have in mind is that the time in percentage should always be given because of its great importance in the preparation of power estimates. A water power which can be developed from a given stream is subject to considerable variation, due to the character of the stream and the climatic conditions, etc., peculiar to each particular catchment area. Hence, each drainage area and each stream may be considered as a law by itself.

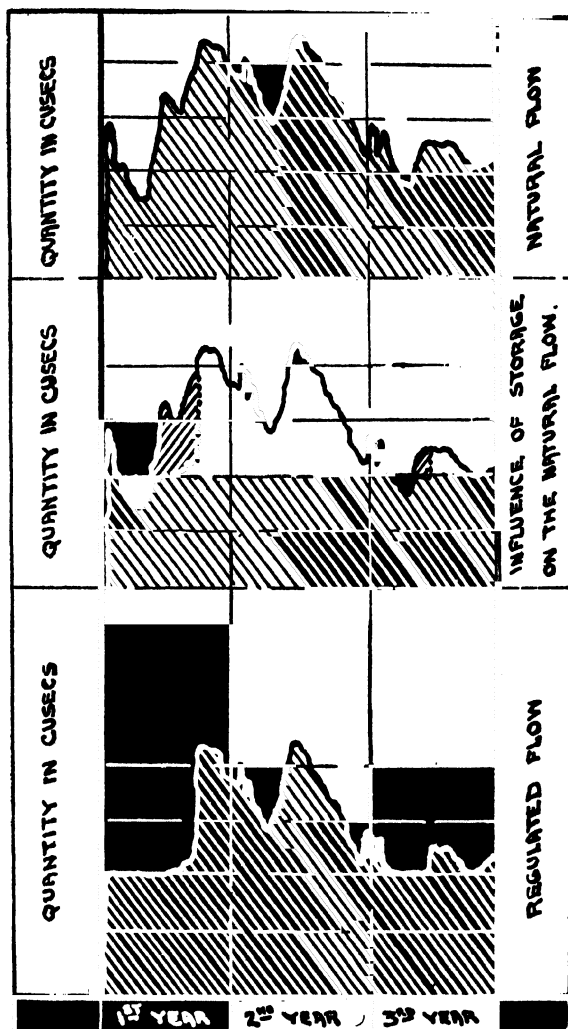
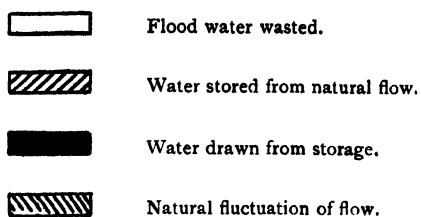


FIG. 5.—Hydrographs showing natural flow, the influence of storage on the natural flow, and possible regulated flow.

A widely used method for studying the effect of storage is by means of the mass or *flow-summation* diagram, *i.e.*, by means of a diagram which shows the net available amount of run-off or supply of water to the reservoir which has accumulated in a given period of time. That is to say, by summing up the run-off—expressed in any convenient unit—after deducting the observed or the estimated evaporation and seepage losses, the increment for a month, week, or day may be used, depending upon the regularity of the stream flow and the run-off records. The outflow, plus or minus storage, shows directly the net inflow. To determine the capacity of reservoirs with the aid of a topographical map, it is first necessary to know the inflow and outflow of water at different stages of the reservoir for different periods of time, the time interval being made as short as possible to eliminate seepage and evaporation losses. In plotting a reservoir capacity curve, values of the quantity of water stored or released, divided by the change in water surface elevation during the period, may be plotted as abscissæ and the corresponding average stage of the reservoir—elevation of water above the outlet gates—and the quantity of water in the reservoir may be plotted as ordinates, and a curve from these values drawn through the points. At any point on this curve the area between the curve and the average stage-height is equal to the capacity of the reservoir at that particular stage. The information required for this method, which determines reservoir capacity, is: time-period; mean gauge-height; inflow area; outflow; quantity of water stored; quantity of water released; changes in the water-surface elevation during the period; and elevation of the water surface above the outlet gates. The accuracy of the determination at different stages will depend upon the accuracy of the observed or estimated quantity of water and changes in the water surface above the outlet gates. The quantity of water impounded in a reservoir can also be determined by multiplying the *vertical* distance between the different contour levels with the *mean* area. Or, by summing up the contents between each of its contours, using the approximate formula:—

$$Q^1 = h/3 (A + a + \sqrt{A \times a}),$$

wherein

Q^1 = storage in cubic feet; h = vertical distance between contours in feet.

A = area of a contour in square feet.

a = adjacent contour in square feet.

Using a simple method, let us take the following example: Reservoir area, as found by planimeter, is 24 to 600 acres for the

different contour levels surveyed to 10-ft. contours, shown as follows :—

Contour Level.	Area in Acres.	Mean Area in Acres.
10	24	(Draft limit)
20	40	32
30	76	58
40	100	88
50	140	120
60	200	170
70	300	250
80	420	360
90	600	510 (full)

The question is to determine the capacity of the reservoir for the different contour levels up to full capacity. For this case, capacity in acre-feet is :—

Mean Area in Acres.	Capacity in Acre-Feet.	Total Capacity in Acre-Feet for Contour Levels.
32	$32 \times 10 = 320$	...
58	$58 \times 10 = 580$	900
88	$88 \times 10 = 880$	1,780
120	$120 \times 10 = 1,200$	2,980
170	$170 \times 10 = 1,700$	3,680
250	$250 \times 10 = 2,500$	7,180
360	$360 \times 10 = 3,600$	10,780
510	$510 \times 10 = 5,100$	15,880
	<u>15,880</u>	

which represents, in theoretical energy, something like

$$15,880 \times 1.025 = 16,277 \text{ kW. for one hour per foot head.}$$

In certain countries (this country included) and in certain territories the value of a given amount of storage is relatively less than in other countries or territories because of the relatively lesser head capable of development. Mountainous countries or territories are more fortunate in this respect, as they offer great differences in elevation within relatively short distances where head can be secured. Obviously, the greater the head capable of development, the greater will be the value of a given quantity of water. Speaking relatively,

storage with high-head developments is of very great value, much greater than for medium and low-head developments.

As shown elsewhere in the text, power from flow is expressed :—

$$\begin{aligned} \text{h.p.} &= (HQ \times 62.4) / 550 = HQ / 8.82 = HQ \ 0.1134 \text{ (100 per cent. efficiency),} \\ \text{kW.} &= (HQ \times 62.4) / 734 = HQ / 11.8 = HQ \ 0.0846 \text{ (100 per cent. efficiency),} \end{aligned}$$

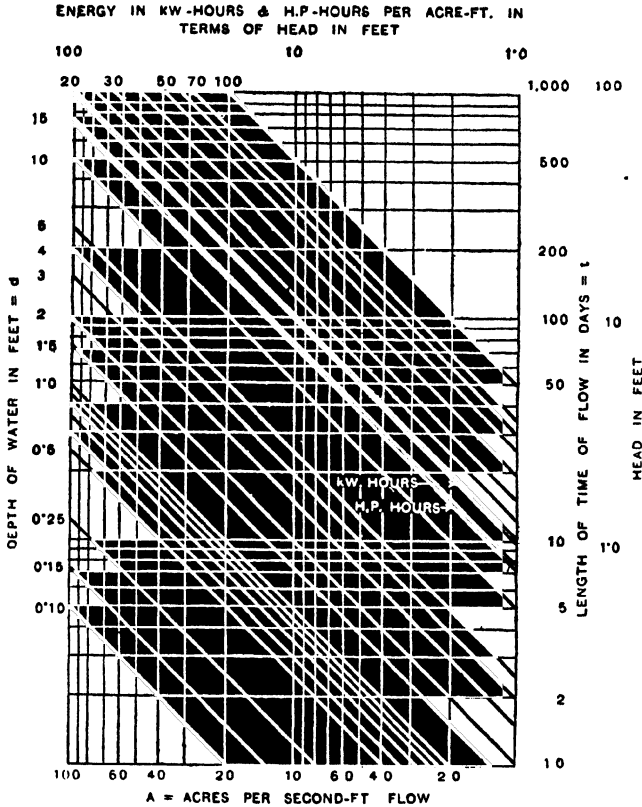


FIG. 6.—Storage, flow, and energy, in terms of depth of water, acres per sec.-ft. flow, and head in feet.

from which we may deduce the energy from the storage of this flow to :

$$\begin{aligned} \text{h.p.-hours} &= HQ / 8.82 \cdot T = HQ / (8.82 \times 60 \times 60) = HQ / 31,680 \\ &= 0.0000315 \text{ HQ (100 per cent. efficiency)} = 0.0000315 \text{ HQ efficiency.} \end{aligned}$$

$$\begin{aligned} \text{kW.-hours} &= HQ / 11.8 \cdot T = HQ / (11.8 \times 60 \times 60) = HQ / 42,500 \\ &= 0.0000235 \text{ HQ efficiency.} \end{aligned}$$

Thus, we may write, h.p.-hours = $e(HQ/k)$, and kW.-hours = $e(HQ/K)$, wherein, for the usual working conditions, we have :—

e	k	K
60 per cent.	52,600	70,900
65 „	48,600	65,300
70 „	45,100	60,000
75 „	42,200	56,700
80 „	39,500	53,000

where Q = cubic feet of water per second (ft.³/sec.).

H = effective or net head in feet.

T = time in hours.

e = efficiency in percentage.

Run-off in acre-feet can be obtained from the equivalent factor, 1.0 cusec per day equals 1.984 acre-ft., or, roughly, 1.0 acre-ft. is equal to 0.5 cusec for one day. Also, 1 acre of water 1 ft. deep equals 43,560 cub. ft. And 1.0 acre of water under 1.0 ft. head will (*theoretically*) produce 1.025 kW.-hours of energy or 1.375 h.p.-hours of energy when drawn down 1 ft. (see Figs. 6 and 7).

Sometimes the problem is to determine how many hours or days it takes to fill a given pondage area to a given depth of water. The time in days of twenty-four hours can be approximately expressed :—

$$T' = Ad/1.984 = 0.5 Ad; \text{ or } d = 1.984 T'/A \text{ and } A = 1.984 T'/d;$$

wherein A = area in acres *divided* by the discharge in cusecs = area in acres per cusec or ft.³/sec.

d = depth in feet.

T' = time in days of twenty-four hours each.

Usually low-head developments depend on a perennial stream flow with sufficient storage or pondage capacity to handle hourly fluctuations of load. In general, day to day or weekly storage for very low-head developments is commercially impossible for a large or fairly large power output, due to prohibitive cost of land and of structural works necessary to impound the vast amount of water to carry over this period. For pondage, if we know the flow of a stream and know that the mean head lasts for nine hours (a fairly long time-period), then if the pondage is of sufficient capacity to handle this flow for one day of twenty-four hours, the turbines can use not only the water flowing for the nine hours' full operation, but also the amount which has flowed into the forebay or regulating basin for pondage during the remaining fifteen hours' inoperation; the available energy could thus be increased nearly $24/9 = 2.7$ times. Hence, we can assume that natural rates of stream flow can be modified by pondage of almost any useful amount; for high-head developments a relatively small amount is of great value. While *storage* means the capacity

which may be available to increase the extremely low flows for several days, weeks, or perhaps months, *pondage* generally means the water-storage capacity which is available for a shorter period of time, such as part of a day of twenty-four hours, and used for the purpose of equalising or regulating the variations in the power load during any one day. The amount of pondage required in any given case depends upon the load factor and character of load, and it may limit the size of plant for any predetermined load factor. Natural conditions at the site, coupled with excessive cost, will often decide whether it is the hourly or daily load factor or the load factor for any longer period of time which determines the amount of pondage. With restricted pondage or no pondage at all, the hydraulic equipment of an independently operated plant must obviously be operated as a *base* plant carrying as much of the load as possible up to its full capacity. Stream flow may not correspond to the variation in power demand during the day, thus necessitating regulated pondage as an important component of the economic regulation of flow. Where pondage is used, flashboards should preferably be provided (for very low-head developments) to raise the normal level of water. The use of flashboards often proves a valuable asset, particularly for *very* low-head plants, because the controlling factor is head variation. As the head falls with the drawing down of the level of pondage water, the power and efficiency of the turbines may fall if the speed is kept practically constant, this being the desired operating condition for constant frequency of alternating current generators and motors. Pondage should always be located as near the power house as possible. Very often excellent storage and pondage facilities are available at relatively less total cost for a given total capacity, but their location is so far upstream or so far from the pressure pipe line, that full advantage cannot always be taken of the available water for economic and dependable regulation. With medium and high-head developments a forebay is usually built for both pondage and regulating purposes. It is sometimes suggested that the forebay should be designed to store a half-day's or a 24-hour supply of water so that the plant can be employed to carry the *base* load running at about 80 per cent. load factor all the time—this would generally be done if the cost and good all-round advantages would warrant it.

The amount of power that can be developed during the period of *lowest* flow, sometimes defined as the amount of energy which is available for every hour of every day of every year, is seldom taken as the amount upon which hydro-electric developments are based. This power basis (modified in form) is usually defined as the *primary power*. For every water-power development it does not necessarily signify the amount of power for 8,760 hours of the year.

3. Leakage and waste, etc. ; from 10 per cent. to as much as 40 per cent. may have to be allowed for this. The latter value would be that of a badly-located, poorly-designed, constructed, and operated hydro-electric system.

Modifications due to pondage or/and forebay capacity have already been mentioned, but it may again be stated that it usually means the amount of water-storage capacity available hourly or daily to equalise the variations in the power load, and it also represents the retention of the flow of a stream during hours of light load and for use during hours of heavy load. Hence, not unlike storage in one respect (which represents the retention of flood-water for use during times of scarcity), the study of pondage must follow and not precede the power market investigation, which will definitely settle questions pertaining to the character of the future loads, such as the most probable daily or weekly, etc., load factors. The load factor, as referred to here, meaning the ratio of the average to the maximum power during a certain period of time, as a day, week, month, or year. Usually, in making power estimates and determining power capacities from stream flow, it is sufficiently accurate to assume full load conditions for a part of the day corresponding to the load factor, and to assume no load during the remaining part of the day ; that is to say, a 60 per cent. load factor is equivalent to a full load for 14.4 hours, and a 30 per cent. load factor for 7.2 hours. The diversity of the system load is important and should also be determined. Diversity factor is taken as the ratio of the sum of the maximum power demands of the subdivisions of any system to the maximum demand of the whole system or part of the system to the maximum demand of the system under consideration, measured at a predetermined point—the more non-coincident these peak services are, the greater will be the diversity factor. A close study of the power market is very essential in determining the possible percentage utilisation of the water power available during the first few years of a new development, with particular reference to the period of building up the load.

The rate of flow ; the amount of water required from the forebay or from pondage (as the case may be) ; the maximum draw-down ; total pondage available ; the net available head, and the total cost, etc., are matters which usually settle the size of and the best operating conditions for pondage, whether for high, medium, or low-head developments. The amount of pondage required in any given case will depend upon the load factor, or the amount of pondage may limit the size of the plant for any predetermined load factor—depending, of course, on the type of development. In the determination of forebay or pondage capacity, one of three conditions can be assumed, these being :—

1. A condition when the average daily rate of flow *equals* the daily rate of flow which is completely equalised by pondage.
2. A condition when the average daily rate of flow *is less* than the daily rate of flow which is completely equalised by pondage.
3. A condition when the average daily rate of flow *is greater* than the daily rate of flow which is completely equalised by pondage.

That is to say, the average rate of flow available during the working hours can be expressed

$$Q = Q_h/Y; \text{ also } Q = Q_p/Y;$$

the average rate of flow during the working hours supplied from pondage can be expressed

$$Q_p' = Q_h (1 - Y) \quad Y = Q_p (1 - Y)/Y = Q_d/Y,$$

wherein Q_d is the equivalent average 24-hour rate of flow in cusecs which the forebay would yield with no flow into the forebay; Q is the average rate of flow available during the working hours; Q_h is the average 24-hour rate of flow; Q_p' is the average rate of flow during the working hours supplied from pondage; Q_p is the 24-hour rate of flow which is completely equalised by pondage; and Y is the load factor.

When the average rate of flow is greater than Q_p , then

$$Q = Q_h + Q_p' = Q_p = Q_h (1 - Y)/Y; \text{ and } Q_p' = Q_d/Y,$$

but, when the average 24-hour rate of flow is smaller than the 24-hour rate of flow, which is completely equalised by pondage, then

$$Q = Q_h/Y; \text{ and } Q_p' = Q_h (1 - Y/Y).$$

Example.—A forebay for a low-head plant is to be built mainly for regulating purposes, and its estimated dimensions are 877 yds. by 2,000 yds. The maximum draft-head is 5 ft., and the net head on the hydro-electric plant is 76 ft. It is desired to determine the power and energy output when drawing on the forebay (not the conduit) for (a) one day of twenty-four hours; (b) for 8.45 hours of one day. Due to leakage and waste of water, and actual daily variations in load factor, etc., we shall take a coefficient of 0.85, which makes a reduction of 15 per cent. in the total energy. The over-all hydro-electric efficiency (up to the switchboard) is taken at 73 per cent.

$$\text{Area of pond} = (877 \times 3) \times (2,000 \times 3) = 15.75 \text{ million sq. ft.,}$$

and the storage in cubic measurement for a 5-ft. draw-down is

$$15.75 \times 5 = 78.75 \text{ million cub. ft.}$$

As 1 cusec equals 0.0864 million of cub. ft. in one day of twenty-four hours, we have $78.75/0.0864 = 916$ cusecs for twenty-four hours'

working; but, for 8.45 hours' operation the same draw-down is equal to

$$(24 \div 8.45) \times 916 = 2.84 \times 916 = 2,620 \text{ cusecs.}$$

One acre-ft. equals 43,560 sq. ft., then, in terms of acres, we have $78,750,000/43,560 = 1,807$ forebay dimensions.

As 1 acre under 1 ft. head equals 1.025 kW.-hours if drawn down 1 ft., then for 5 ft. draw-down (assuming 73 per cent. hydro-electric efficiency, up to the switchboard), we have

$$5.0 \times 1,807 \times 1.025 = 9,269 \text{ kW.-hours.}$$

When working under 76 ft. head, we have

$$76 \times .73 \times 9,269 = 514,250 \text{ kW.-hours}$$

for one hour, or $24 \times 514,250 = 12.340$ million kW.-hours; if the permissible draft is used in 8.45 hours, we have

$$\text{kW.} = [(2.84 \times 916) \times 76/11.62] \times .73 = 12,290,$$

but $2.84 \times 12.342 = 35.0$ million kW.-hours;

making the total available kW.-hours

$$= 0.85 \times 35 = 29.75 \text{ million kW.-hours.}$$

Now, let load factor be 35 per cent., then, for a 24-hour rate of flow of 916 cusecs, we have (for 35 per cent. load factor):—

$$Q_p = Q_h/(1 - Y) = 916/(1.0 - 0.35) = 1,400 \text{ cusecs,}$$

in which Q_p equals the average rate of flow during the working hours supplied from pondage; Q_h equals the average 24-hour rate of flow; Y is the load factor (decimal). That is to say, this capacity will completely equalise all daily flows at 35 per cent. load factor up to 1,400 cusecs. However, we are dealing with a quantity above 1,400 cusecs, hence, for 78.75 million cub. ft. capacity, we have

$$Q_d = 78.75/0.0864 = 916 \text{ cusecs.}$$

The average rate of flow available during the working hours in this case is

$$Q = Q_h/Y = 916/.35 = 2,620 \text{ cusecs,}$$

and the average rate of flow during the working hours *supplied from the forebay* is

$$Q_p' = Q_h(1 - Y)/Y = 916 \times (1.0 - 0.35)/0.35 = 1,701 \text{ cusec,}$$

but $916 + 1,701 = 2,620 \text{ cusecs.}$

At 73 per cent. hydro-electric efficiency let kW. generated at the switchboard equal QH 0.059, then, in this case,

$$\text{kW.} = Q(76 \times 0.059) = Q 4.48 = 2,620 \times 4.48 = 11,740 \text{ kW.};$$

the total available kW.-hours at 35 per cent. load factor is expressed

$$\text{kW.-hours} = \text{kW.} \times 8,760 \times .35 = 11,740 \times 8,760 \times .35 = 36,000,000 \text{ kW.-hours,}$$

and, allowing for water efficiency, we have

$$0.85 \times 36 = 30.6 \text{ million kW.-hours.}$$

Storage or secondary power is often based on the continuous available power for about 100 days up to perhaps 180 days (six months) of the year, the period of time varying with the amount of storage, dependability of water, the probable use of the power, the diversity of demand and the load factor of the system, etc. Different to secondary power, primary power (which is for continuous service) always demands a higher degree of insurance; in fact, the former class of power (neglecting interconnection of plants) is rarely sold with a guarantee. Moreover, it simply represents the amount which is developed above the primary power and therefore available only

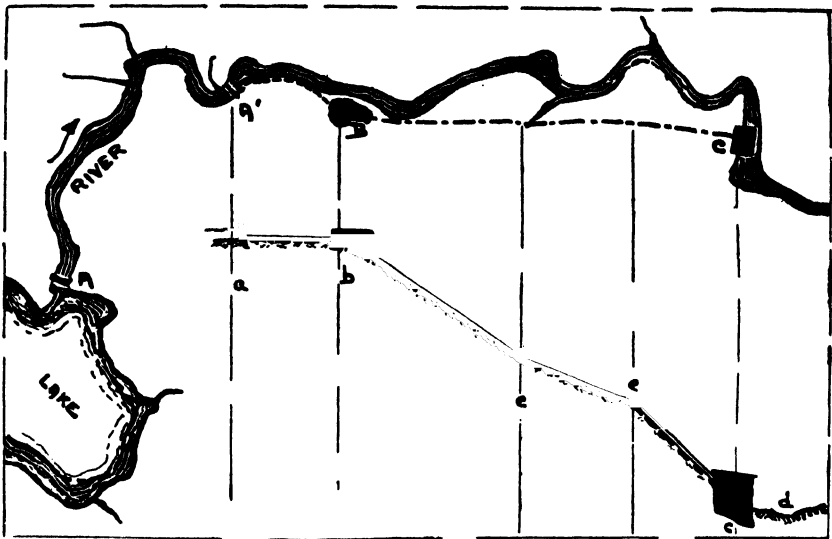


FIG. 8.—Showing water-power development with storage, pondage capacity, and pipe line, all favourably situated.

for intermittent service, this being the chief reason why it can be sold cheaper. Very often the amount usually known as secondary power is many times that of primary power, and can be effectively utilised for industries not requiring continuous plant operation, or it can be used very effectively in connection with electrical operation of plants situated on streams of different run-off characteristics. This class of power may also be represented by the excess of generating rating over that required for primary power, and can be delivered in varying amounts on most days of the year.

As already understood, the size of reservoirs is limited principally by insufficient storage capacity and by cost. Conditions may, however, require that the reservoir be operated to utilise practically all the

available storage each year without any special regard for maintaining a definite minimum regulated flow ; or conditions may exist where it is necessary that a predetermined minimum flow be maintained, the aim being to utilise the available storage to increase the dependable flow. Perhaps the object would be to obtain, as near as possible, the highest minimum flow and at the same time use each year practically the whole volume of available stored water. The two methods more often employed to increase primary power and *dependable* flow are: (a) Utilisation of available storage to secure the maximum possible increase of the *lowest* flow or extreme low-water flow over a period of time ; (b) utilisation of available storage to secure the maximum increase in the *ordinary* minimum flow or ordinary low-water flow. The variation in stream flow (from flood to lowest flow) is usually so great that it invariably pays to install hydro-electric plant capacity sufficient to utilise much more than the estimated low-water flow of the stream. Flood and low-water extremes can be modified by the retardation of precipitation in its movement toward the streams ; the principal agency being porosity of the soil. The retardation of flow in one form or another will generally help to increase dependable flow, *i.e.*, the maximum and uniform power which can be depended upon at all times in all or most years. Apart from retardation of precipitation due to porosity of the earth, regulated flow from reservoirs is *the* principal agent for mitigating flood and low-water extremes of streams.

The capacity of the material of the water-bearing stratum to transmit water depends mainly on the effective size of grain and on the porosity of the water-bearing material. Experiments have shown that the flow of water through any material varies nearly as the square of the effective size of grain. The flow of water is also greatly increased with an increase in porosity. The water table is defined as the surface below which any extended body of porous material is completely filled with water. The water table of an underground reservoir corresponds to the surface of an ordinary surface reservoir. The underground reservoir of streams usually sustains the normal low-season flow for several weeks during a total absence of precipitation, and the rate of depletion is often very low. It is known that streams frequently discharge an inch or more per month from the catchments without addition of precipitation ; this is more particularly true of the summer period when (in some countries) rainfall seldom affects the ground water reservoirs. In the study of the percentage relation of run-off to rainfall, it is sometimes found to vary materially in two years having practically the same total precipitation ; the factor principally responsible for this variation is the relative height of ground water level, and in order to ascertain the amount added or drawn from

ground water reservoirs or storage, it is first necessary to collect such information as may be available regarding the general changes in height of water surface in existing wells, marshes, and so forth. This part of water storage study and investigation is very important, and, unfortunately, is nearly always uncertain in character (see p. 30).

As regards storage of water in the earth, percolation and capillarity are of special consideration in that they bear a relation to the important factor of run-off. Peat and muck soils have a high normal water content due partly to their humus composition, which gives them a greater affinity for water than the common material soils, and partly to the large relative pore space accompanied by a high degree of capillarity. Part of this pore space is in the loose character of the soil itself and part in the cellular structure of the muck. Part of the water held by saturated muck is released readily by drainage, and about an equal amount is quickly reabsorbed by drainage soil when moisture is supplied by rainfall. Soil water-storage, with particular reference to this class of soil, may roughly be summed up in the following words: (1) Storage capacity of the soil under favourable water table conditions will permit taking advantage of evaporation to assist drainage; (2) value of this storage varies directly as the depth of soil above the table water; (3) storage capacity of muck soil is a valuable drainage factor; (4) soil storage tends to equalise run-off, and this effect will extend throughout the hydraulic system of waterways; (5) deep soil above the water table is a good protection against flood, etc.

The rate at which muck, silt, etc., is deposited in a reservoir depends upon the quantity carried by the stream, the size of the reservoir, the volume of stream flow, and the character of the catchment area. If the capacity of the reservoir is small compared with the average annual discharge of the stream, it will act in the nature of a retention basin, and when the stream is in flood and carries its greatest load of muck or silt, a portion of the muck or silt goes over the dam. On the other hand, if the reservoir capacity is large compared with the annual stream flow, little goes over the dam, and the muck and silt brought down by the stream is deposited into the reservoir.

It is becoming quite common practice to install bye-pass valves in water-power plants in order to have a regulatable means of passing water through the plant when the quantity being discharged through the waterwheels is below normal. This is necessary where several plants are on the same river or where water for irrigation must be provided in addition to the discharge of the plant. Under very low-heads the requirements are satisfactorily met by the various forms of slide gates, but high-heads present great difficulties. It is not

easy to generalise on this point because the size of the valve is an important factor, but it is safe to say that only comparatively small slide gates will operate satisfactorily under heads in excess of about 50 ft. It is difficult, often impossible, to operate slide gates, and they are subject to vibration and erosion of the most destructive nature. It is now generally conceded that a needle valve is the valve most successfully used for this service. No other valve will discharge water in a smooth jet at all openings, and, unless the discharge is free from disturbance, vibration will certainly result.

In general, there are four arrangements of the regulating apparatus in connection with conduits and tunnels for storage dam-outlets which may be used :—

1. A conduit through the dam with a regulating valve installed at the entrance to the conduit on the upstream face of the dam. Such a valve usually consists of a needle-shaped plug pointing toward the mouth of the conduit and enclosed in a cylindrical structure from or set into the face of the dam. The conduit leading from the valve receives the water at spouting velocity and carries it through the dam to the downstream face. The valves are submerged and inaccessible for maintenance and repairs unless it is possible to draw down the reservoir and unwater them (see p. 49).

2. A supply well is provided in the dam structure in which water is admitted through slide gates and from which it is discharged through a conduit having a regulating valve located at the entrance to the conduit. The slide gates are located at several different elevations in order that they may successfully operate under low pressure as the water level in the reservoir is drawn down.

3. The same as in (2) except that the regulating valve located at the discharge end of the conduit discharges directly into the air.

4. A conduit through the dam with a regulating valve located at the discharge end of the conduit on the downstream face of the dam.

Whereas the valve located at the downstream end of the conduit is accessible for inspection and repairs, the valve at the upstream end is never accessible except when it is possible to draw down the reservoir. If a valve gets out of order it cannot be repaired until the low-water season. For this reason it is customary to place batteries of valves at different levels to be sure that some of them will be regularly unwatered by the seasonable variations of level in the reservoir. Furthermore, the upstream type of valve cannot be protected from debris and, on account of the high velocity at which water is drawn into it, it is very likely to get plugged in such a way that the valve will be put out of commission.

Example.—A river in much demand, with an average discharge of 200 cusecs (second-ft.), is to supply a regulating basin (forebay); the pondage has an area of 132 acres. The problem is to find how many days, or part of a day, it would take to fill or empty this area to a depth of 3 ft. The ratio $A'/Q = 0.66$.

First obtain the value A'/Q = area in acres per cusec, then, from Fig. 6, follow up the vertical line for 0.66 acre per cusec until the diagonal line d (3 ft.) is intersected. Then, for this example, move to the left of chart to 6.6 acres and follow up the diagonal line for 3 ft., we find the flow in days to be 1 day (twenty-four hours) for 0.66 acre, or 10 days for 6.6 acres per cusec, that is, the time for filling is—

$$T' = 0.66 \times 3 \times 0.5041 = 0.998 \text{ day, say 24 hours.}$$

If we desire to know what the depth in feet will be for an average discharge of 200 cusecs (second-ft.) from a stream flowing continuously for twenty-four hours or longer, the answer can be obtained from a glance at Fig. 6 or from the formula:—

$$d = \frac{T' \times 1.9835}{A} = \frac{0.998 \times 1.9835}{0.66} = 3 \text{ ft.}$$

Or, if we know the draft-head and have assumed the length of time, then the area in acres for pondage can be found from Fig. 6 or from the formula—

$$A = \frac{T' \times 1.9835}{d} = \frac{0.998 \times 1.9835}{3} = 0.66 \text{ acre per cusec,}$$

making $0.66 \times 200 = 132$ acres (pondage).

Other important uses may be made of this chart.

Example.—The catchment area of a certain district, planimeted from map, is 86,600 sq. miles; the mean discharge during a month of thirty days is 37,400 cusecs (second-ft.). Determine the *gross* run-off in terms of acre-feet.

From Fig. 7 we find for thirty days' discharge (see left of diagram) at the rate of 3.76 sec.-ft. (follow diagonal line from left of diagram downward to the intersection of thirty days) is 224.4 acre-ft. Therefore 37.6 sec.-ft. (see further to left of diagram) is 2,244 acre-ft. Hence, 37,600 sec.-ft. or 10,000 times is—

$$10,000 \times 224.4 = 2,244,000 \text{ acre-ft.}$$

From Fig. 7 we also find that one acre-ft. under one foot-head (commencing at the bottom right-hand corner of the chart) gives 1.025 kW.-hours, hence we obtain per foot-head a gross annual energy output for 100 per cent. water efficiency of—

$$2,244,000 \times 1.025 = 2.3 \text{ million kW.-hours,}$$

that is, for one foot-head and 2,244,000 acre-ft. we follow the diagonal P, until the value (shown at bottom of chart) is intersected when to the right of chart we find the desired kW.-hour value.

Other important uses may be made of this chart.

Useful Conversions

Reservoirs and Storage—

1 acre = 43,560 sq. ft.
1 acre-ft. = 1 acre of water 1 ft. deep.

Thus the amount of water required to cover one acre to given depth :—

Depth in Inches and Feet (Acre Inches and Acre-Feet).	Cubic Feet Contained in One Acre to Depths Given.
1.0 in.	3,630
2.0 "	7,260
4.0 "	14,520
6.0 "	21,780
8.0 "	29,040
10.0 "	36,300
1.0 ft.	43,560
1.2 "	50,820
1.4 "	58,080
1.6 "	65,340
1.8 "	72,600
2.0 "	87,120

1 acre-ft. = 0.504 cusec for 24 hours.
1 sq. mile = 640 acres.
1 " = 27.88 million sq. ft.
1 sq. mile-ft. = 1 sq. mile of water 1 ft. deep.
= 0.75 cusecs for 12 months (nearly).
= 2.25 " 4 " "
= 3.0 " 3 " "
= 90.00 " 1 month "

0.1 million cub. ft. stored will give 1.16 cusecs for 24 hours.
" " " 2.25 " 12 "
" " " 9.25 " 3 "
" " " 27.75 " 1 hour.

1,000 million cub. ft. stored will give 30 cusecs for 1 year.
" " " 40 " 9 months.
" " " 60 " 6 "
" " " 90 " 4 "
" " " 120 " 3 "
" " " 360 " 1 month.

1 cusec per sq. mile = 0.03719 in. depth of rainfall in 24 hours.
= 1.041 " " " 28 days.
= 1.116 " " " 30 "
= 1.156 " " " 31 "

1 cusec will discharge = 61.49 acre-ft. in month of 31 days.

$$\begin{aligned}
 &= 59.5 \quad \text{,,} \quad \text{,,} \quad 30 \quad \text{,,} \\
 &= 55.54 \quad \text{,,} \quad \text{,,} \quad 28 \quad \text{,,} \\
 &= 1.983 \text{ acre-ft. per day.} \\
 &= 0.5385 \text{ million of gals. per day.} \\
 &= 15.08 \text{ million of gals. in 28 days.} \\
 &= 16.15 \quad \text{,,} \quad \text{,,} \quad 30 \quad \text{,,} \\
 &= 16.69 \quad \text{,,} \quad \text{,,} \quad 31 \quad \text{,,} \\
 &= 2.678 \text{ million cub. ft. in 31 ,,} \\
 &= 2.592 \quad \text{,,} \quad \text{,,} \quad 30 \quad \text{,,} \\
 &= 2.419 \quad \text{,,} \quad \text{,,} \quad 28 \quad \text{,,} \\
 &= 0.0864 \quad \text{,,} \quad \text{,,} \quad 1 \text{ day}
 \end{aligned}$$

$$\begin{aligned}
 1 \text{ ft.}^3/\text{sec.} &= 1 \text{ cusec} = 6.23 \text{ gals. per second.} \\
 &= 538,453 \text{ gals. per day.}
 \end{aligned}$$

$$\begin{aligned}
 1 \text{ million gals. per day} &= 1.86 \text{ cusecs.} \\
 &= 3.68 \text{ acre-ft.}
 \end{aligned}$$

Square feet area of a circular basin = (gals.)/(depth in feet $\times$ 6.23).

Number of gals. contained in a rectangular basin is
length $\times$ breadth $\times$ depth (in feet) $\times$ 6.23.

Millions cub. ft. $\times$ head (in feet) $\times$ 17.5 = kW.-hours.

Millions cub. ft. $\times$ head (in feet) $\times$ 24 = e.h.p.-hours.

1 h.p. is the power required to raise 33,000 lbs. 1 ft. in 1 minute
= 33,000 ft.-lbs. per minute.
= 1,980,000 ft.-lbs. per hour.

Thus, taking Q = cubic feet of water discharged per second ;
 G = weight of 1 cub. ft. of water = 62.4 lbs. ;
 H = total head in feet.

Then gross power in ft.-lbs. per second = QHG (see p. 8).

Gross h.p. = $QHG/(33,000/60) = 0.1134QH$.

And, for discharge in cubic feet per minute, we have—

$$QHG/33,000 = 0.00189QH.$$

Rate of Draft from Reservoir.—For any decrease in elevation of a reservoir level the rate of draft can be expressed—

$$Q' = \frac{QH}{(H' - h)}$$

where

Q' = draft in ft.³/sec. ;
 Q = average draft in ft.³/sec. ;
 H' = normal head in feet when full ;
 H = average head in feet ;
 h = fall in level below H in feet.

CHAPTER V

STREAM FLOW

THE stage of a river or stream is usually taken as the height of the surface of the water in the river * or stream with respect to datum, zero, or bench-mark. For any particular site it is important that the information be based on the available power with respect to this datum or bench-mark ; particularly on measurements of discharge made at high- and low-water stages at the particular site. A river or stream is the channel for the discharge of the available precipitation on the drainage area.

In general, it is unsafe to develop power for the maximum flow of the stream. Many conditions indirectly influence the maximum as also the most economical power development point. On the other hand, hydraulic works (the dam or weir in particular) are nearly always designed and laid out with the highest possible stream flow in mind.

At the present time the ordinary minimum stream flow is frequently taken as the safest basis upon which to draw up water-power estimates. In general, the more important problem is to determine the proper or the economical amount above the ordinary stream flow for which the development should be built. Numerous independent variables enter into the determination of this amount. The correct amount will depend, in no small way, upon the degree of insurance or continuity of supply justified by the conditions the particular project and the load, etc., offers. If absolute continuity of service must be guaranteed and must be included in the agreement for energy supply, then for such conditions, and, assuming little or no storage, uncertain facilities for dependable storage, and no possibility of interconnection with other plants, it would generally be advisable to take the ordinary minimum† flow and to use it in connection with the maximum load anticipated. Very often the development of the stream only to a minimum flow entails such a great expense for a relatively small amount of power output as to render a project based on such a flow entirely uneconomical (see p. 25).

* Throughout the text the word *stream* will be used in place of the two words *river* and *stream*.

† The averages of minimum flow for the lowest two consecutive seven-day periods in each year, over a period of years.

The basin of a river or stream is the expanse of country, bounded by a winding ridge of high ground, over which the rainfall flows down towards the river traversing the lower part of the valley. River basins vary in extent according to the configuration of the country, ranging from the insignificant catchment areas of streams rising on high ground very near the coast and flowing straight down into the sea, up to immense tracts of great continents, where rivers, rising on the slopes of mountain ranges far inland, have to traverse vast stretches of valleys and plains before reaching the ocean.

The rate of flow of rivers depends mainly upon their fall. The fall of a river corresponds approximately to the slope of the country it traverses; and as rivers rise close to the highest part of their basins (generally in hilly regions) their fall is rapid near their source and gradually diminishes, with occasional irregularities, till, in traversing plains along the lower part of their course, their fall usually becomes quite gentle. As the velocity of a stream increases with its fall, the size of a channel conveying a definite average flow varies inversely with the fall, and the depth inversely with the width. The discharging efficiency of a river or stream within the limits of its bed depends on the fall and the cross-section of the channel, and the only way of increasing the fall is to reduce the length of the channel by substituting straight cuts for the winding course.

In obtaining a longitudinal section of a stream or river it is usual to plant a line of stakes following the sinuosities of the river already chained and levelled. The cross-sections are referred to the line of stakes, both as to position and direction. The determination of the surface slope is sometimes a difficult matter, partly from its extreme smallness and partly from oscillation of the water. It may be advisable to take the slope in lengths of about 2,000 ft. by four simultaneous observations, two on each side of the channel. The banks of the channel (river or stream) or sides of a valley are distinguished as the right and left bank respectively, the person looking down the valley or downstream when defining which bank is to be developed, etc.

The two most important stages of a stream to be known are the extreme low-water level and the extreme high-water or flood level. In some countries the former condition is very rarely known because : (1) a river rarely reaches its lowest level ; (2) there are alterations in the land as to forest and other growth ; (3) river obstructions ; (4) increased use of water further upstream ; (5) varied conditions of discharge, etc. In water-power investigations usually the lowest levels of which records can be found is taken as the approximate low-water level. Generally in the tropics the rise of floods is greater than in temperate regions. The rise of a river in flood should be

taken with respect to that particular reach of the river as the flood level varies in different parts of its course. The highest flood level is usually taken from the best available records. Flood water flow, to be really worth conserving, should be available at least once in three or four years. When no reservoir exists, the volume of continuous supply from any catchment area is limited to the minimum, or extreme dry weather flow of the stream draining it. This flow depends entirely upon the soil and rock to store water in the particular area, and to yield it continuously to the stream by means of concentrated springs or diffused seepage. If, however, a reservoir is employed to equalise the flow during and before the period of dry weather, the minimum flow continuously available may be increased to a much higher value, depending upon the capacity of the reservoir in relation to the mean flow of the stream supplying it (see p. 64).

Streams continue to flow in the summer months, even when there is no rainfall, due to the storage in the ground, and the only time that the flow of the stream is approximately proportional to the rainfall is when the ground is full of water, or, in some climates, during the period lasting from two to six months. Sometimes a period of full ground water may last only two months in the entire year, and in some cases it may last six months, especially during a period when the rainfall is large and evenly distributed in the winter and spring months. It is obvious that the most valuable water power is that which is capable of delivering continuously the nearest to the maximum power which can be delivered at any time.

Records (showing overestimation or failure) are available to prove that, in the investigation of stream flow, the engineer sometimes failed to apply a reliable run-off coefficient or failed to use the best suited method in estimating the annual run-off in the absence of gaugings, or he could not get reliable stream flow records extending over a sufficiently long and continuous period, or he was inclined to give favour to a higher amount of water power or rely too much on certain records of stream flow not exactly reliable and safe. In Chapter II., several methods are given for estimating the run-off of a catchment in default of stream gaugings; these different methods should be referred to, as they deal with: (*a*) the rainfall percentage method; (*b*) the rainfall loss method; (*c*) the empirical formulæ method; and (*d*) the direct comparative method. Every year has its period of relatively low water with one or more days of least flow (perhaps total drying up), but the average rates of discharge during the low periods of some years, as well as the least daily discharge, are much greater than in others. These absolute minimum values are of great importance; their significance should not be overlooked by the investigating engineer, and they should be mentioned when

preparing estimates and determining the amount of power available from the given catchment and stream (see p. 192).

For a stream, the wetted perimeter is taken as the length of the line outlining the bottom and sides. For a uniform flow the grade or slope line has a constant slope or fall which can be measured by the sine S of the angle of inclination, that is, $S = h/L$; wherein h = head lost in length L . The hydraulic radius $R = A/w$ = cross-sectional area/wetted perimeter. For a stream we should take *mean* depth as area/width.

On streams fed mainly by ground water, the low-water flow is determined by the precipitation several months previously. The best ground water supply is often found in regions of ample precipitation where both soil and subsoil are of fine sandy character and forest cover is absent. Clay soils retard percolation or seepage and facilitate surface run-off. Deep sandy soil and subsoil will permit a large amount of percolation and will quickly carry the percolating water to depths from which it is safe against return by capillary action. Rock outcrops, muck, clay soil, and steep slopes assist in a rapid concentration of flood water. The determination of the amount of ground water and the water table is usually attended with doubt and uncertainty (see p. 64).

Methods for measuring stream flow are many and varied. At the present time the current meter method is the one most extensively used for the different flowage conditions and stream sections. With this method the most accurate results are obtained when the hydraulic section on which measurements are taken is selected primarily with regard to flowage conditions, *i.e.*, a stream with smooth sides and bottom, and the reach or section uniform for a sufficient distance both up and down stream to ensure a smooth flow and avoid turbulence; these flowage conditions are also very favourable to the use of and the accuracy of slope measurements. For the current meter method it is important that the current meter be calibrated before use and also after use, *i.e.*, calibrate the meter for straight-line flow and choose a reach or section so as to have the meter working under conditions where reasonably straight and smooth flow can be expected to occur for different stages of the stream. For the different channel conditions, the most satisfactory gauging methods to use are shown in Table A.

The methods for measuring stream flow can be divided into the following groups: (1) direct discharge methods; (2) mean velocity-area methods. The former (1) includes all methods whereby the total discharge is determined directly independent of any knowledge of the velocity or sectional area, and embraces such means as (a) weirs; (b) chemical gauging; (c) volumetric or bulk method;

(d) gravimetric method. Of the four methods of measurement, the most reliable and one of the most used methods is by means of the weir, *i.e.*, a vertical obstruction placed in the stream bed containing a notch of either trapezoidal, rectangular, or "V" form over which the water falls. The *standard* sharp-crested weir is the type used extensively in most countries. A *standard* type consists of a metal crest (of steel plate) with a sharp right-angle corner on the upstream edge, a crest width of $\frac{1}{8}$ in. and bevelled to an angle of 45° on the downstream face. Knowing the dimensions of the weir, the observation necessary for the determination of the discharge is the head or the depth of water flowing over the crest. It is important that the channel of approach be straight and uniform, and that the crest be carefully levelled. As in most other matters, the "personal equation" is an important factor; as an instance, if the head on the weir is not carefully measured, serious inaccuracy is sure to arise.

TABLE A.—SHOWING THE MOST SATISFACTORY GAUGING METHODS

Conditions of Stream.		Gauging Method Recommended.
Small streams	— — —	Current meter or sharp-crested weir.
Open channels of uniform section	—	Current meter or rod float.
Narrow, deep, and uniform section	—	Current meter, rod float, colour or brine method.
Wide and shallow streams	— —	Current meter.
Deep and narrow streams	— —	Current meter or chemical process.
Swift or turbulent mountain streams		Chemical process, and sometimes the current meter.

Many experiments have been made with weirs of different forms, resulting in a number of different formulæ for weir discharge. All of these formulæ are empirical and require the use of coefficients derived experimentally; consequently the accuracy of the results depends upon the accuracy of the experiments and the use of the formulæ and coefficients under conditions identical with those under which the original experiments were made. Weirs carefully used under favourable circumstances are accurate within about 1 per cent.

A weir is the most satisfactory method of measuring the flow of water in open channels when the obstruction of the channel and change of water levels, due to the interposition of the weir, are not objectionable. The sharp crested rectangular weir without end contractions, known as the suppressed weir, in which the sides of the channel form the ends of the weir, is the basis of the formulæ $Q = cL(H\sqrt{H})$; where Q is the quantity in cusecs; L the length of weir or notch in feet; and H is the difference in level of surface of

water upstream and the crest or sill of the weir in feet. The table of values of (c) for various heads (H) and heights of crest (p) is :—

TABLE IV.—VALUES OF c IN $Q = cL'H \sqrt{H} = cL' \frac{2}{3} \sqrt{H^3} = cL'H^{3/2}$

Head in Feet.	Height of Crest (p) above Bottom of the Channel of Approach in Feet.									
(H)	4	5	6	7	8	9	10	11	12	13
1.0	3.376	3.356	3.344	3.335	3.329	3.325	3.322	3.317	3.314	3.311
1.2	3.391	3.366	3.350	3.339	3.332	3.326	3.322	3.316	3.311	3.308
1.4	3.409	3.378	3.359	3.346	3.336	3.330	3.324	3.316	3.311	3.307
1.6	3.429	3.393	3.370	3.354	3.343	3.334	3.328	3.319	3.312	3.308
1.8	3.350	3.408	3.382	3.363	3.350	3.340	3.333	3.322	3.315	3.309
2.0	...	3.425	3.394	3.373	3.358	3.347	3.338	3.325	3.317	3.311

Weirs of a length extending approximately twenty times the head (except in cases where the velocity of approach is extremely low); or weirs of moderate crest-length having high velocities of approach; or those in which the velocity of approach is irregularly distributed, or in which the leading channel is subject to action of the wind,

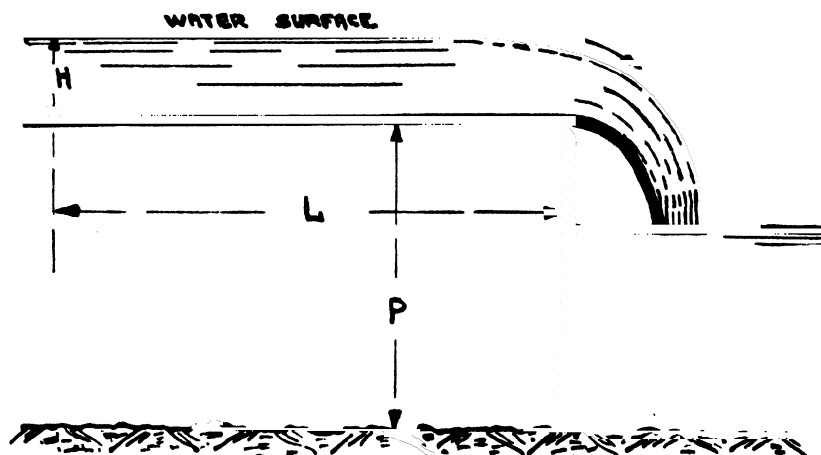


FIG. 9.—Diagrammatic view of weir.

should either be subdivided into a number of sections or the head should be observed not only at both sides, but also at intermediate points across the channel of approach. The elevation of the crest should be measured at short intervals of its length in determining the zero readings of the hook gauges.

A weir should, if possible, be located so that smooth flow, free

from eddies, surface disturbances, or the presence of considerable quantities of air in suspension, exists in the channel of approach. Where stilling racks are used they should be arranged to give approximately uniform velocity across the channel of approach. The uniformity of velocity should be verified by current meter.

One method of determining the head on a weir or the elevation of the surface of the flowing water is by direct observation upon it as it flows. The head on the weir may also be observed by hook gauges placed in stilling boxes communicating with orifices approximately 1 in. in diameter in the sides of the channel of approach, approximately 1 ft. below the level of the crest, the head being observed independently at both sides of the channel. The head (H) on the weir should be measured at a distance of not less than five nor more than ten times the thickness of the stream going over the weir upstream from the weir. The weir should be sharp crested, with smooth vertical crest wall, complete crest contraction, and free overfall; and, as explained elsewhere, the crest to use should be of metal free from rust, with sharp right-angle corner on the upstream edge, a crest width of $\frac{1}{8}$ in. and bevelled to an angle of 45° on the downstream face. The crest should be carefully levelled. The side walls of the channel should be smooth and parallel and should extend downstream beyond the overfall above the level of the crest. The head on a weir must be carefully measured and the *mean* level of the water surface must be measured with equal care.

Formulae much used in determining the discharge in ft.³/sec. over weirs of the rectangular, trapezoidal, and the "V" notch types, are:—

Rectangular Weirs—

$$Q = 3.33 (L - h/5) h^{3/2} \text{ (with double end contraction)}$$

or
$$Q = 3.33 (L - 0.2 H) [(H + h)^{3/2} - h^{3/2}]$$

$$Q = .0801 (L - H/60) H^{3/2} \text{ (without end contraction).}$$

Trapezoidal Weirs (Cippoletti)—

$$Q = 3.367 L h^{3/2}$$

$$Q = .081 L H^{3/2}$$

"V" Notch (Right Angle) Weirs—

$$Q = 2.54 h^{3/2}$$

or
$$Q = 2.48 H^{2.47} \text{ (vertical angle } 90^\circ)$$

or
$$Q = 0.0611 H^{3/2}$$

where Q = cusecs = ft.³/sec.

h = head or height over crest or sill, in feet.

H = head in inches.

L = length of crest or sill, in feet.

The object of the *Cippoletti* weir is to eliminate the effect of end contractions. This weir has a top width of $1.3L$ with bottom width of L , and a depth of $.6L$ with side slopes of 1 in 4. This type of weir is widely used in various parts of the world.

The method sometimes used for taking approximate measurements is to place a board across the stream at some point which will allow a pond to form above. The board should have a notch cut into it with both side edges and the bottom sharply bevelled toward the intake. The bottom of the notch, which is called the crest or sill of the weir, should be perfectly level and the sides vertical. In the pond back of the weir, at a distance not less than the length of the notch, a stake should be driven near the bank, with its top precisely level with the crest. By means of a rule or a graduated stake, the depth of water over the top of the stake is measured, making allowance for capillary attraction of the water against the sides of the weir. For greater accuracy this depth may be measured to thousands of a foot by means of a hook gauge. Having ascertained the depth of water over the stake then calculate the amount of water flowing over the weir; using the following table:—

TABLE V.—TABLE OF WEIR MEASUREMENT

Head (H) in Inches.	(A)	1/4 (B)	1/2 (C)	3/4 (D)
0	...	.05	.14	.26
1.0	.40	.55	.73	.92
2.0	1.13	1.35	1.58	1.82
3.0	2.07	2.34	2.61	2.90
4.0	3.20	3.50	3.81	4.14
5.0	4.47	4.81	5.15	5.51
6.0	5.87	6.25	6.62	7.01
7.0	7.40	7.80	8.21	8.63
8.0	9.05	9.47	9.91	10.35
9.0	10.80	11.25	11.71	12.17
10.0	12.64	13.12	13.60	14.09
11.0	14.59	15.09	15.59	16.11
12.0	16.62	17.15	17.67	18.21
13.0	18.74	19.29	19.84	20.39
14.0	20.95	21.51	22.08	22.65
15.0	23.23	23.82	24.40	25.00
16.0	25.60	26.20	26.80	27.42
17.0	28.03	28.65	29.28	29.91
18.0	30.54	31.18	31.82	32.47

Note.—(*H*)=depth of water over weir in inches. *Column (A)*=discharge in cubic feet per minute for whole inches in depth. *Columns (B) to (D)*=discharge in cubic feet per minute in whole inches and fractions. *Thus*, capacity of stream =cubic feet per minute $\times$ width of weir in inches.

In this approximate method there are certain proportions which must be observed in the dimensions of the notch. Its length or width should be between four and eight times the depth of water flowing over the crest of the weir. The pond back of the weir should be much wider than that of the notch, and be of sufficient width and depth that the velocity of flow or approach be not over one foot per second.

The above table shows the number of cubic feet passing for each inch or fractional inch in depth (H) and for 1 in. in width. *Example*: Say the weir or notch is to be 24 in. wide, and water measures 6 in. from the top of stake, that is, the depth of water over the stake is 6 in. Table shows that for each inch in width the flow will be 5.60 cub. ft., or for 24 in. width 134.4 cub. ft.; making $134.4/60 = 2.24$ cusecs. If the depth or head (H) is 9 in., then from table we have 10.80 cub. ft., and for 24 in. is $10.8 \times 24 = 259.2$ cub. ft.; making $259.2/60 = 4.32$ cusecs. For 12 in. depth we have 16.62 or 398.8 cub. ft. for 24 in. width, making 6.64 cusecs flowing over the crest of the weir.

Another important method (applicable to streams and conduits) is the chemical process. This method consists of putting some chemical, usually salt, in the stream (or in the water conduit and pen-stocks) at a certain definite rate, and then at some other point down stream (or in the tail-race), after a uniform mixture has been effected, samples are drawn off and the degree of dilution is determined. In other words, if y lb. of salt be added each second to a stream whose discharge is Q cusecs and after a thorough mixture it is found that x lb. of water contains 1 lb. of salt, the total discharge Q will roughly represent $xy/62.4$. The salt is introduced as a solution of about 16 lbs. per cub. ft. of water, and the rate should be such that the water of the stream after admixture contains not less than 1.0 part in 75,000 of salt. Sufficient of the solution should be used to give a uniform supply for from twenty to thirty minutes. The accuracy of the results depends on the samples being truly representative of the water at the collecting station. At the present time there is a need for some simple and accurate way of measuring water to hydro-electric stations by an electrical method, and the salt-velocity method, based on the fact that salt in solution increases the electrical conductivity of water, should meet such conditions.

The flow measurements of *group 2* can be divided into the following:—

(A) Methods whereby the mean velocity of the entire stream can be determined directly by one operation, such as measurements by (a) slope determination; (b) colour or brine methods; (c) travelling screen; (d) orifice or nozzle methods; (e) Venturi meter, etc.

TABLE VI.—TABLE GIVING VALUE OF $H^{.32} = \sqrt[.32]{H^3 = H \sqrt{H}}$
(To be used with formula for Table IV.)

H	.0	.10	.20	.30	.40	.50	.60	.70	.80	.90
0	.0000	.0316	.0894	.1643	.2530	.3536	.4647	.5858	.7155	.8531
1	1.0000	1.1537	1.3145	1.4830	1.6505	1.8371	2.0238	2.2164	2.4149	2.6189
2	2.8284	3.0432	3.2631	3.4882	3.7180	3.9526	4.1927	4.4365	4.6868	4.9385
3	5.1961	5.4581	5.7243	5.9947	6.2692	6.5479	6.8305	7.1171	7.4074	7.7020
4	8.0000	8.3018	8.6074	8.9166	9.2285	9.5459	9.8698	10.189	10.516	10.846
5	11.180	11.520	11.858	12.196	12.550	12.897	13.258	13.610	13.975	14.331
6	14.697	15.065	15.437	15.812	16.192	16.575	16.955	17.343	17.734	18.120
7	18.520	18.924	19.324	19.724	20.133	20.540	20.950	21.367	21.875	22.210
8	22.627	23.056	23.488	23.910	24.350	24.781	25.222	25.663	26.156	26.550
9	27.000	27.455	27.910	28.361	28.820	29.282	29.747	30.211	30.678	31.151
10	31.623	32.111	32.570	33.060	33.538	34.027	34.513	35.000	35.495	35.985
11	36.483	36.985	37.483	37.999	38.494	39.000	39.500	40.018	40.548	41.040
12	41.569	42.090	42.610	43.145	43.675	44.192	44.725	45.255	45.800	46.333
13	46.872	47.418	47.958	48.506	49.055	49.600	49.534	50.707	51.266	51.824
14	52.383	52.944	53.508	54.075	54.644	55.219	55.786	56.368	56.936	57.514
15	58.094	58.676	59.262	59.846	60.439	61.022	61.614	62.206	62.803	63.400
16	64.000	64.600	65.200	65.809	66.413	67.024	67.632	68.245	68.853	69.473
17	70.092	70.714	71.333	71.954	72.584	73.210	73.835	74.463	75.095	75.730
18	76.367	77.005	77.643	78.284	78.930	79.570	80.216	80.865	81.514	82.169
19	82.819	83.472	84.130	84.789	85.450	86.110	86.772	87.440	88.771	88.771
20	89.442	90.114	90.790	91.463	92.140	92.819	93.496	94.180	94.862	95.548
21	96.234	96.922	97.614	98.303	99.000	99.690	100.39	101.09	101.79	102.49
22	103.19	103.89	104.600	105.30	106.00	106.70	107.41	108.15	108.85	109.59
23	110.30	111.04	111.73	112.48	113.19	113.90	114.63	115.38	116.12	116.80
24	117.57	118.35	119.06	119.79	120.51	121.25	122.00	122.75	123.50	123.25

(B) Methods whereby the total cross-sectional area is divided into sections whose mean velocities, areas, and discharges are summed up to give the total discharge of the stream ; these methods differ only in the manner of determination of mean velocity of the section, and they may comprise the use of one or more of the following means : (a) current meter ; (b) rod float ; (c) surface or sub-surface floats, of which (b) is one ; and (d) Pitot tubes, etc.

One of the most common errors in the measurement of discharge Q is caused by erroneous soundings. Soundings should be taken close enough to indicate and include irregularities in the cross-section. At a number of points the depth should be taken, and from these different depths the mean depth computed. To obtain the cross-section it is not sufficient to take the measurements at one point only ; several cross-sectional areas should also be taken at intervals along the course or reach of the stream being measured. It is usual to plot all soundings for the same mean stage, then the hydraulic section is laid out to some convenient scale and divided into squares or sections ; after the cross-section is accurately determined, it is laid off in vertical sections and enough point stations located on the centre line of each section to enable a consistent vertical curve to be plotted.

The *discharge of a stream* is expressed by : $Q = AVk$ (ft.³/sec.), wherein A = average cross-sectional area in square feet ; V = mean velocity in feet per second ; k = coefficient, varying under different channel conditions. In general, the coefficient will vary inversely with the slope or velocity or roughness, directly as the depth, and, for fairly narrow streams, directly as the width of the stream. The area A may be calculated by the *general* formula for discharge per second :—

$$Q = A_1V_1 + A_2V_2 + A_3V_3 + \dots + A_nV_n,$$

wherein $A_1A_2A_3$, etc., are the component parts of the cross-section, and $V_1V_2V_3$, etc., the mean velocity in each of these parts.

The *average* cross-sectional area A should always be used in the determination of Q , the discharge Q being the product of the average cross-section A and mean velocity V . The final value of discharge Q is computed from the individual discharge measurements, and curves made up from the corresponding mean velocities and the cross-sectional areas for each measurement and the gauge height h' ; the latter should always be available, as the area A is usually determined by soundings with reference to the stage of the stream and the observed gauge heights. By taking measurements at many stages of flow a rating curve can be plotted ; by means of this it is only necessary to read the gauge, compare the reading with the rating curve, and then locate on the diagram the discharge corresponding to the given stage.

Such a diagram is very useful, and, slightly modified, it may also be used to indicate the power available for any given stage.

Many methods are in use for determining stream flow *velocities* by current meters and floats. Some of these methods are :—

(a) By current meter, take the velocities at equal vertical intervals at about 12 in. or more, and obtain their arithmetical mean.

(b) Hold the current meter at each predetermined point for a sufficient time to allow the varying velocities of a cycle at that point to register. Under uniform velocity conditions the duration of time at each point should be not less than one minute. The top or sub-surface station should be just under the water surface, and the lower or bottom station should be as near the bottom of the stream as the meter will operate satisfactorily. The vertical velocities are then plotted and the areas planimetered in order to get the mean velocity, then a horizontal curve is plotted from the mean velocity values. By continuing the curves to the sides and bottom of the stream, the entire section can be considered in the determination of the quantity Q .

For the determination of discharge from a series of current meter observations, the graphical method proposed by *Harlacher* is much used. Let ABC be the cross-section of a river for which a complete series of current meter observations have been taken. Let $1d$, $2d$, $3d$, etc., be the verticals at different points of which the velocities were measured. Let the depths at $1d$, $2d$, etc., be set off as vertical ordinates, and, in these vertical ordinates, suppose the verticals are set off horizontally at their proper depths as shown in Fig. 10 (see small diagram). Thus, if V is the measured velocity at the depth h on the vertical marked $2d$, then $BC_2 = h$, and $AB = V$. Then C_2 is a point in the vertical velocity curve for the vertical cd , and, all the velocities for that ordinate being similarly set off, the curve can be drawn as shown. Suppose all the vertical velocity curves $1d$, $2d$, $3d$, etc., thus drawn correspond to velocities of $1y$, $2y$, $3y$ ft. per sec. ; then BC_2 is the depth at which a velocity of $2y$ ft. per sec. existed on the vertical $2d$, and if BC_2 is set off at $2d$ it gives a point in a curve passing through points of a section where the velocity was $2y$ ft. per sec. If we set off on each of the verticals all the depths thus found, then curves drawn through the corresponding points on the vertical are curves of equal velocity. The discharge of the river may be regarded as a solid having its cross-section as a base. The curves of equal velocity may therefore be considered as contour lines of the solid whose volume is the discharge of the stream.

Let A be the area of the cross-section of the river, $A'A''$ the areas contained by the successive curves of equal velocity (if these cut the surface of the stream, by the curves and the surface). Let Y be the difference of velocity for which the successive curves are drawn.

Then the volume of successive layers of the solid body whose volume represents the discharge, limited by successive planes passing through the contour curves, will be $0.5Y(A + A')$, $0.5Y(A' + A'')$, and so on. The discharge is therefore :

$$Q = Y[0.5(A + A_n) + A' = A'' + \dots + A_{n-1}].$$

In the determination of river discharge from surface velocities it is first of all desirable to choose a reach with good section of the stream which is uniform in depth and width, etc. Then choose a cross-section of the part of the stream near the centre of the reach and plot the cross-section as shown in Fig. 10. That is to say, let ABc be the cross-section, *i.e.*, AB the water surface and $Acc_1c_2c_3 \dots B$ the bed of the stream. Take the surface velocities at points 1, 2, 3, etc., and then plot them as $1d$, $2d$, $3d$, etc. ; these verticals will represent the positions of the soundings and gaugings. Now, draw in the velocity curves $Ad_1.d_2 \dots B$ through the points $Ad_1.d_2 \dots B$; thus dividing the water surface into many equal parts as shown with verticals interconnecting the vertical curves at $Ad_1.d_2 \dots B$ at the points shown. The ordinates thus drawn will represent the surface velocities at these points. At each point multiply the velocity by the depth of water, and plot the product of the two and draw the curve as shown. Calculate the area (or find it by planimeter) and multiply it by a coefficient varying from 0.7 to 0.9 according to the character of the river, the result will give the discharge of the stream. We thus see that the curves of equal velocity are worked out from current meter observations and the discharge is calculated from the contour curves. By this graphical (gauging) method several interesting curves may, for the same river cross-section, be marked in. For instance, if not too complicated, we may give a graphical representation showing :—

- (a) A curve of magnitude of the surface velocity.
- (a') A curve of magnitude of mean velocity in each vertical.
- (b') A mass curve.
- (c') A curve of mean velocity of the whole section ; that is, a velocity which corresponds to the mean velocity of the stream and which passes through the points on the cross-section where the velocity is identical with the mean velocity of the stream.
- (d'') Curve of depth of mean velocity in each vertical ; that is, the depth at which the mean velocity for each vertical can be found (see Fig. 10).

And (f') : curve of the position of maximum velocity in each vertical may be drawn through the points at which the maximum velocity is found.

TABLE VII.—TABLE GIVING VALUE OF $R^{2/3} = \sqrt[3]{R^2} = R^{.67}$
(Showing Values of $R^{2/3}$ to be used with Manning's Formula)

R	.0	.1	.2	.3	.4	.5	.6	.7	.8	.9
0	.000	.215	.342	.448	.543	.630	.711	.788	.862	.932
1	1.000	1.065	1.129	1.191	1.251	1.310	1.368	1.424	1.480	1.534
2	1.587	1.639	1.691	1.742	1.792	1.842	1.891	1.939	1.987	2.034
3	2.080	2.126	2.172	2.217	2.261	2.305	2.349	2.393	2.435	2.477
4	2.519	2.561	2.603	2.645	2.686	2.726	2.766	2.806	2.846	2.886
5	2.924	2.964	3.002	3.040	3.078	3.116	3.154	3.192	3.229	3.266
6	3.302	3.339	3.375	3.411	3.447	3.483	3.519	3.555	3.590	3.625
7	3.659	3.695	3.725	3.764	3.798	3.832	3.866	3.900	3.933	3.967
8	4.000	4.034	4.067	4.100	4.133	4.165	4.198	4.230	4.262	4.294
9	4.326	4.359	4.491	4.423	4.455	4.486	4.518	4.549	4.580	4.612
10	4.642	4.674	4.704	4.735	4.765	4.795	4.836	4.866	4.886	4.916

(c) By current meter, take the velocity at 0.6 of the water depth. By the use of this method with bottom and sides of smooth and uniform section for a sufficient distance both up and down stream (for normal stream flow of moderate depth) the use of a coefficient is unnecessary in the determination of the mean velocity. This method is usually reliable over a wide range of conditions, and in North America it is in common use.

(d) By current meter, take the velocity at 0.2 and that at 0.8 of the water depth and obtain one-half the sum. This method is used for general stream gauging. When closer accuracy is required, two current meters should be used, one for the method (c) and one for method (d). The degree of accuracy will depend on the ratio of width to depth and the velocity.

(e) By current meter, take the velocity at 0.2, 0.6, and 0.8 depth. The mean velocity can be obtained by *dividing* by four the sum of the measured velocities at 0.2 and 0.8 depth *plus* the velocity at 0.6 depth. The mean velocity can also be computed from the *approximate* formula :—

$$V = \frac{k(V' + V'' + V''')}{2 + V''''}$$

V' = surface velocity.

V'' = velocity at 0.2 of the depth.

V''' = velocity at 0.6 of the depth.

V'''' = velocity at 0.8 of the depth.

k = coefficient varying from 0.85 to 0.95, according to the depth, width, and velocity.

(f) By current meter, take the velocity near the water surface and use from 0.80 to 0.95 of the result The value of this coefficient

will depend on the depth of the water, its velocity, and the nature of the bed of the stream. The point of observation should be from 6 to 12 in. below the surface, its proper location depending on the depth of the stream. This method is generally used in swift flowing streams and in streams during times of freshet.

(g) By current meter, take velocity at 0.5 depth. The *approximate* formula for mean velocity is : $V = V' \times 0.96$, wherein V' = velocity at mid-depth.

(h) By current meter, take velocity in the vertical line by uniformly lowering and raising several times the current meter throughout the range of water depth. This method may also be used to determine the coefficients given in some of the above-mentioned methods.

(i) By current meter, take six intervals of depth, giving seven velocities, including the surface velocity V_1 and the bottom velocity V_7 , the mean velocity in the vertical is then computed from formula :—

$$V = \frac{1}{18} [V_1 + V_7 + 4(V_2 + V_4 + V_6) + 2(V_3 + V_5)]$$

(j) *By means of floats.* The different kinds of floats are surface, sub-surface, or a combination of the two, and rod floats. Nearly all surface floats are useless, as they give only the surface velocity, and with almost any wind blowing the velocity cannot be measured with accuracy. Also in general the sub-surface float is unreliable, as it is easily caught by eddies and cross-currents which move it about. A good approximate value for average velocity is to multiply the surface velocity by 0.85. One of the most satisfactory surface floats is the ordinary bottle, sealed up, with a weight placed in the bottom. The most satisfactory surface float is the *rod float*. The rod float can be made of wood or metal tubing so weighted that, when placed in the water, the rod remains vertical with not more than a few inches of its length appearing above the surface. The mean velocity, by means of the rod float, is computed by the *approximate* formula : $V = V^o [1.0 - 0.116(\sqrt{D} - 0.1)]$,* wherein V = mean velocity in vertical ; V^o = observed rod float velocity ; D = proportion of depth not reached by rod. The mean or average velocity can also be *approximately* determined from the formula : $V = A(L/t)k$, wherein L = length in feet measured ; t = average time in seconds ; k = coefficient = 0.75 to 0.85 ; A = calculated cross-sectional area.

* Francis-Lowell hydraulic experiments give :—

$$V = V^o [1.012 - 0.116(\sqrt{D + D'})]$$

where D' = depth of the stream.

D = clearance between the bottom of the rod and the bed of the stream.

This clearance should be less than 0.25 D' .

For *closed conduits* the following methods of measurements are used : (a) salt solution (chemical process) ; (b) Venturi meter ; (c) Pitot tube. For *open conduits* the methods of measurements in use are : (a) weir ; (b) current meter ; (c) chemical process. For conduits it is considered good if the measurements come within 1 to 3 per cent. of the true discharge, and for streams it is considered good if measurements come within 4 to 9 per cent. of the true stream discharge when using the current meter for normal stage on a section with reasonably good flowage conditions.

The operation of a current meter very often differs greatly from that by which the meter is rated, thus introducing errors. That is to say, when *rating* a meter it is rigidly attached to a rod and both are moved through a *vertical* plane, but when *operating* a meter it may or may not be held in the vertical position. Unless the size of a meter is negligible as compared with the cross-section of the stream, inaccuracies must also result, since the immersion of a large meter in a relatively small stream not only changes the direction of currents, but it modifies their effects. A curve for the ideal meter would be a horizontal line indicating an identical number of revolutions for all velocities. With most commercial meters it is found in practice that the number of revolutions suddenly drops off at the lower velocities.

The *slope measurement* is associated with an empirical formula and the friction factor or coefficient of roughness, commonly known by the symbol η , is always associated with stream flow. In the determination of the friction factor from the cross-section and measured slope, etc. (whether effective or not), there is cause for probable inaccuracy due not only to the roughness of the stream bed and banks, but to an inefficient cross-section and slope as well as to the engineer's judgment of channel conditions. The value of η may also differ widely between flood and low flow of the stream. For flood flow the total cross-section and slope may be effective in producing velocity, but for a low flow a considerable part of the slope in certain reaches or sections of the stream may be lost—particularly in the wide and shallow sections—with the result that the measured cross-section used in calculating the value η may often show a discrepancy compared with the same section at other parts of the reach and/or for different stages of the stream. For stream flow computation, using slope measurement, the writer favours the *Manning* formula, not only because of its simplicity as compared with the more complicated Kutter formula, but also because of the application of Kutter's coefficients of roughness which are well known, and which can be used in this formula to give results practically identical. Manning's formula is expressed :—

$$V = cR^{2/3}S^{1/2} = cR^{.67}S^{.5} = c^{\sqrt[3]{2}}\sqrt[3]{R^2} \sqrt{S}; \text{ and } Q = cAR^{2/3}S^{1/2}, \text{ wherein } c = 1.4867/\eta.$$

For nearly all except very low discharges the value η in this formula is a fairly good constant. Kutter's formula gives too large values for small slopes. In using the Kutter formula it is customary to assume a value for η for a stretch of stream channel and to use the same value regardless of change in slope or in hydraulic radius. Also, in fixing the value of η the character of the channel alone is considered in the Kutter formula, the slope usually being disregarded so far as its effect on η is concerned. The values of η to be used in the Manning formula for calculating values of c in $V = (1.4867/\eta) R^{2/3} S^{1/2}$, are :—

TABLE VIII.—SHOWING VALUES OF η TO BE USED FOR DIFFERENT CONDITIONS OF CHANNEL

	Value of c .	Value of η .	Conditions of Channel.
1	29.7	0.0500	For large streams in flood ; mountain streams.
2	49.5	0.0300	For moderate size streams with loose stone bottoms, relatively shallow, with very irregular course and sharp bends.
3	59.6	0.0250	For streams (not large) with smooth sandy bottoms, relatively narrow and deep, with irregular channel.
4	66.0	0.0225	For earth sections (conduits) in fairly good condition.
5	74.3	0.0200	For earth sections (conduits) with bottom and sides well trimmed ; masonry lined or cobble stone section.
6	87.5	0.0170	Canals with gravel bottoms and sides well rammed.
7	99.0	0.0150	Rock-cut and tunnel section fairly smooth (good).
8	114.3	0.0130	Tunnels and other sections, concrete or timber lined (good).

For the different character of channels, values of η will vary somewhat from the values given above. For instance, (1) and (2) in the above table may be greater but rarely less. Also, the value of 0.025 may be high for the *ordinary* earth section conduit. For rock-cut unlined sections, the value of η will usually lie between 0.025 and 0.030, but this will depend on the amount of trimming. Also, for tunnels (concrete-lined) η may be 0.0125 or even 0.0145.

The two following examples, taken from actual practice, may help to give a better understanding of the importance of the variable factor η for streams :—

Observation of (A) section.

Channel, straight for about 750 ft. ; *banks*, both side-slopes very irregular ; *cross-section*, shows considerable variation ; *bottom*, very

rough and uneven ; *average depth of water* at centre is 14 ft. ; *average surface width* of water 60 ft. At a given stage, discharge of stream is 715.1 cusecs ; hydraulic radius $R = 5.45$ ft. ; coefficient $\eta = 0.0367$; slope $S = 0.0003$; and mean velocity $V = 2.27$ ft. per second.

Observation of (B) section of the same stream.

Channel, very irregular ; *banks*, very irregular (roots, trees, and brush on both sides) ; *cross-section*, very large variations ; *bottom*, very irregular and full of holes ; many logs, etc., impeding flow. At the same stage, discharge of stream is 715.1 cusecs ; hydraulic radius

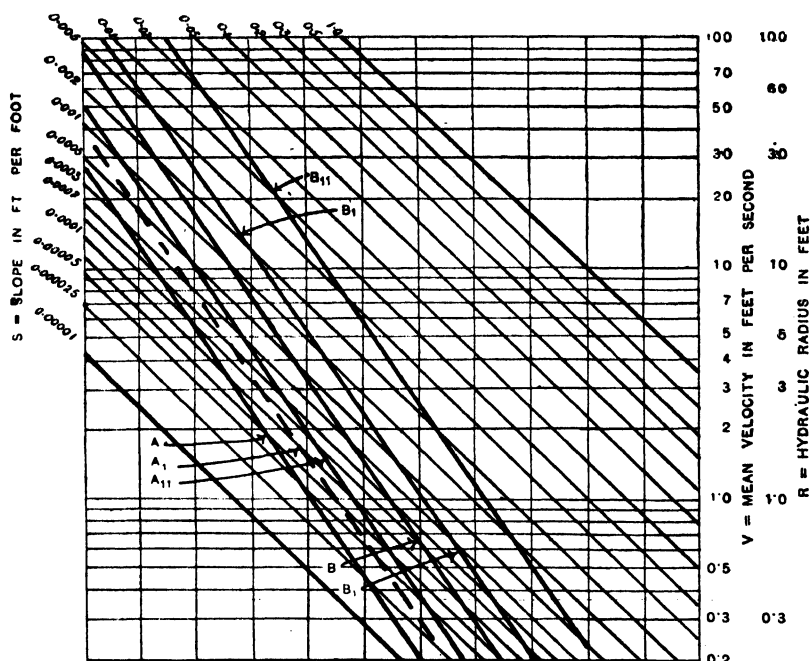


FIG. 11.—Values of V , R , S , and η for flow of water in streams, earth conduit, sections, rock-cut conduit sections, concrete-lined tunnels, and pipes.

Values of coefficient of roughness η are given for the following conditions :— A , very smooth pipes ; A_1 , ordinary pipes ; A_{11} , rough pipes ; A_1 , tunnel and rock-cut (concrete-lined) ; B , tunnel and rock-cut (well trimmed) ; B , earth conduit section (good) ; B_1 , earth conduit section (irregular and rough) ; B_1 , streams (small) in good condition ; B_{11} , streams in very bad condition.

$R = 5.68$ ft. ; coefficient $\eta = 0.145$; slope $S = 0.00221$; and mean velocity of flow = 1.9 ft. per sec. These values of η are seen to be widely different from (1) and (2), which goes to show the many possible inaccuracies likely to occur when making use of the slope measurement for irregular or doubtful channel conditions. If,

for these two examples, we take equal values of $R = 5.45$ ft., and $S = 0.0003$, then we get the following relations :—

	(A)	(B)
Coefficient of roughness — — —	0.0367	0.146
Mean velocity of flow — — —	2.27 ft. per sec.	0.61 ft. per sec.
Discharge of stream — — —	715.1 cusecs.	192.7 cusecs.
Relative capacity of section — —	100 per cent.	27 per cent.

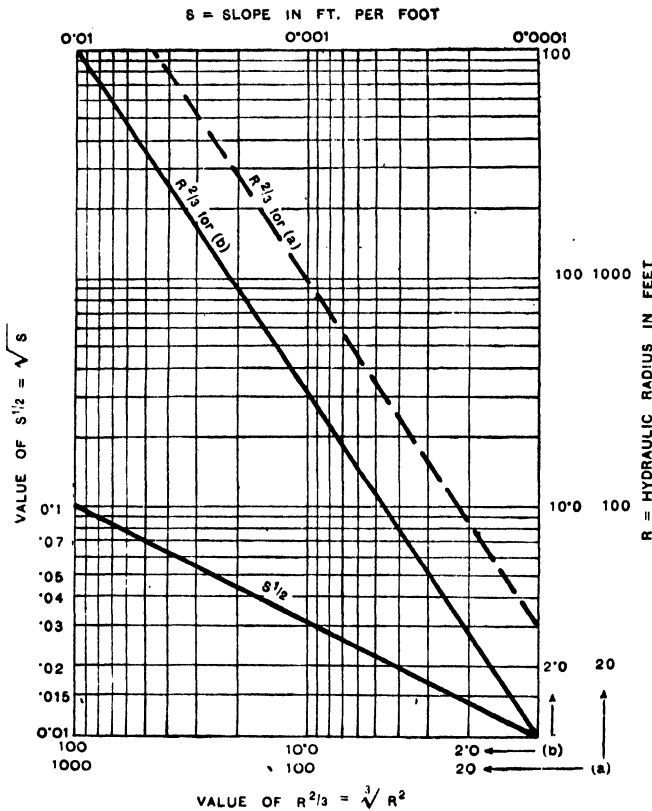


FIG. 12.—Values of $R^{2/3}$ and $S^{1/2}$ in Manning's formula.

Thus, due to the relative difference in channel conditions (character of stream-bed and banks), we have, for the same stream, a difference in capacity of 73 per cent. This example helps to illustrate the bad effect of high η and to show the great advantage of keeping the friction factor as low as possible. Certainly channel conditions

can be changed or new courses can be dug, but Nature can be interfered with within certain limits only, and these limits require very careful investigation and study to properly understand the ever-changing regimen and behaviour during flood flow and for all other conditions of the stream flow.

Given any three variables of the values R , η , and S , for the channel conditions given, the other value V can be determined almost at a glance from Figs. 11 or 15. That is to say for Fig. 11—

1. *Given* R , η , and S , follow the horizontal value of R to the intersection of η , then vertically to S ; at this intersection the value V is obtained as read at the right of diagram.

2. *Given* S , η , and V , follow opposite direction commencing at V , the value R being read on the right of diagram. For example—

2a. *Given* S and V , the value η being assumed, follow the horizontal value of V to the intersection of S , then move vertically to the assumed value of η , when to the right is found value R .

3. *Given* R , η , and V to find S , follow the horizontal line of value R to the intersection of value η , then from value V follow horizontally to the slope values until the vertical value of S meets the value η —the value S is the desired slope.

4. *Given* S , R , and V to find η , follow the value of V horizontally to S , then from value R move horizontally until the vertical value S meets the horizontal value V ; η is found at the point of intersection of S and V . (Note that η is for A , A' , A'' , and B , B' , B'' .)

It will be noted that the value η in Fig. 11 has its basis from the general conditions of the different channels mentioned in Table VIII.

Take the following example, which is for a developed waterway (concrete-lined conduit) of a water-power development :—

Let	Sectional area of conduit = 30 sq. ft. = A ,
	Conduit grade = 0.00129 ft. per ft. = S ,
	Length of conduit = 1,000 ft. = L ,
	Coefficient of roughness = 0.013 = η (see A_1 of Fig. 11),*
	Hydraulic radius = 1.924 ft. = R ,

then, using formula :

$$V = cR^{2/3}S^{1/2} = (1.4867/\eta)^3 \sqrt{R^3} \sqrt{S} = (1.4867/\eta) R^{0.67} S^{0.5},$$

we have from formula (or from Fig. 12) :

$$R^{2/3} = (1.924)^{2/3} = 1.55 \text{ ft. and } S^{1/2} = (0.00129)^{1/2} = 0.03593,$$

$$\text{then, velocity } V = \frac{1.486}{0.013} \times 1.55 \times 0.03593 = 6.4 \text{ ft. per sec.}$$

$$\text{and discharge } Q = cAR^{2/3}S^{1/2} = AV = 30 \times 6.4 = 192 \text{ ft.}^3/\text{sec.}$$

In gross power, per 1,000 ft.-head, this is equivalent to

$$\text{h.p.} = QH 0.1134 = 192 \times 1,000 \times 0.1134 = 21,772.$$

* This value of η is for relatively small size and smooth concrete-lined sections.

Unless a definite statement is made as to the actual available power up to a given point in a water-power system, the amount stated is either meaningless or ambiguous. In other words, for a given amount of water, the *gross* horse-power value should ordinarily commence and finish at the diversion or head-works intake to the developed waterways, and the horse-power value to decrease in amount all along the system of waterways, etc., up to the electric distribution or the consumer's service. In estimating available power, use can be made of the following constants :—

TABLE IX.—SHOWING THE EFFICIENCY CONSTANT FOR h.p. AND kW. TO BE EXPECTED AT DEFINITE POINTS IN THE HYDRO-ELECTRIC SYSTEM.

Conditions.	System Power Constants.	
	h.p.	kW.
(a) At the head-works or intake — —	HQ 0.1134	HQ 0.0846
(a') Gross or theoretical power — —	HQ 0.1134	HQ 0.0846
(b) At the turbine shaft — —	HQ 0.0907	HQ 0.0677
(c) At the generator switchboard — —	HQ 0.0855	HQ 0.0636
(d) At the end of the transmission line — —	HQ 0.0790	HQ 0.0586
(e) At the receiving-station switchboard — —	HQ 0.0760	HQ 0.0563
(f) At the consumer's terminals — —	HQ 0.0700	HQ 0.0518
(g) At the l.p. distribution service — —	HQ 0.0630	HQ 0.0466

To *g* this would be equal to an *over-all* efficiency of nearly 50 per cent. Hence a good ordinary value to assume for estimating purposes would be around 60 per cent. *over-all* efficiency up to the bulk power consumer or 75 per cent. to switchboard (c).

Useful Data

1 cub. ft.	= 6.22786 gals. (Imperial).
	= 7.476 gals. (U.S.A.).
1 cub. in.	= .0036 gal. (Imperial).
	= .01638 litre.
1 gal. (Imperial)	= 1.2 gal. (U.S.A.).
	= 4.543 litres.
1 cub. metre	= 1,000 litres.
	= 220 gals. (Imperial) nearly.
	= 35.31 cub. ft.
1 litre	= .03531 cub. ft.
	= 61 cub. in.
	= .220087 gal.

1 gal. (Imperial) at 62° F.	= 10 lbs. in weight.
1 cub. ft. of pure water at 62° F.	= 62.2786 lbs. in weight.
1 cub. in. of pure water at 62° F.	= .03604 lb. in weight.
1 ton of pure water at	= 35.967 cub. ft.
	= 224 gals. (Imperial).
	= 1 cub. metre.

Gallons	=(litres/4.5458)=.219946 litre.
Cub. ft.	=(gallons/6.228)=.1607 gal.
Cub. ft. per sec.	=(gallons per hour/22.42)=.00004457 gal. per hour.
Cub. ft. per min.	=(gallons per hour/373.7)=.002672 gal. per hour.
Litres per sec.	=(gallons per hour/792)=.001262 gal. per hour.
Lbs. per sq. in.	=(feet of head/2.312)=.4325 ft. of head.
Metres	=(feet/3.28)=.304795 ft.
Inches	=(millimetres/25.4)=.0384 mm.

Properties of a Circle—

Diameter × .886226	= side of an equal square.
Diameter × 3.14159	= circumference.
Diameter ² × .7854	= area of circle.
Circumference × .31831	= diameter.
	= $3.5446 \sqrt{\text{area of circle.}}$
Diameter = 1.1283	$\sqrt{\text{area of circle.}}$
$\pi = 3.14159$	= ratio of circumference to diameter.

Temperature—

Centigrade to Fahrenheit and vice versa :

$$C^{\circ} = (5/9) (F^{\circ} - 32^{\circ}) ; F^{\circ} = (9/5) (C^{\circ} + 32^{\circ}).$$

Water expands on freezing 1/11th of its original bulk.

Character of Developments

Water-power plants which have a natural uniform stream flow, and natural *base-load* plants, and those which depend on storage and built to handle relatively low load factors in varying degrees. The lower the load factor for which a plant may be built, the greater its construction cost per average unit of power, and a condition is approached where it pays to operate a steam plant to carry peak loads in conjunction with the hydro-electric plant; the correct division of load between the steam and hydro-electric plants can be worked out only by a study of the laws of economy in each case. Each water-power site has its own peculiar problems, etc., to solve.

CHAPTER VI

THE GRAVITY CONDUIT

THE hydro-electric system consists of three main energy conduits, viz. : (a) the water conduit (open or closed and of the pressure or non-pressure type); (b) the penstocks; and (c) the electric power transmission line. Of these we find the principal economic problems are largely governed by (1) the gradient of the water conduit; (2) the size of penstocks; and (3) the weight of electrical conductors—it being understood that the shorter the distance, the greater the economy in each case. For the three different yet interdependent conduits containing and conducting the energy, the size and resulting cost is a direct function of the maximum amount of energy transmitted *at any one time*; moreover (along the lines of Kelvin's law), each conduit should satisfy the requirement of minimum cost when the sum of the annual cost of conduit *plus* the value of the energy annually lost is a minimum. Obviously every class of conduit carrying energy should seek to satisfy this law.

The first object of a water conduit for water-power development is *head*. Conduits are constructed for the purpose of obtaining a greater head than can be directly obtained at the dam, or weir as it is sometimes called. A head of water is required to produce velocity, and this velocity is necessary to overcome obstructions such as sharp bends, sudden enlargements, and contractions and friction through the conduit. This friction depends on the velocity of flow, the conduit length, its shape, and the character of material forming its construction. Although the very object of a conduit is head, its purpose is to conduct the water from the dam or weir to (a) the forebay or regulating basin; (b) the surge tank or the penstock; or (c) to the power house direct, depending on the particular type of development. When water is to be conducted from a dam to the power house, two or three methods are available for accomplishing this, namely :—

(a) To build a dam at the upstream end of the fall or rapids and conduct the water through a closed conduit (tunnel, etc.) to a surge tank, then through a penstock direct to the power house, thence back to the stream. Or :

(b) To build a dam or weir above the falls and conduct the water, diverted from the stream, by means of an open conduit (canal, flume,

etc.) along the hillside to a receiving point (regulating basin or fore-bay) from which point water is led by a pressure pipe to the power house and stream below. And, for lower heads :

(c) To build a dam at the up or down stream end of the fall or rapids and conduct the water so that the entire head is concentrated at the dam itself, from which point the water is conducted through penstocks or direct from an open flume (as the case may be) to the power house and tail-race and river below.

Topographic and other natural conditions usually decide the type of development. In general, (a) and (b) are for high and medium head developments, but conditions may call for a compromise of one with the other. For instance, experience has proved that the risk of interruptions of service is far less for tunnels and closed conduits in general than for hillside conditions, and apart from a possible increase in first cost, within reason, every consideration should be given to this matter in the location and final selection of conduit routes.

The preliminary survey as well as a fairly detailed investigation and study of the conduit routes should preferably both be made at the same time. The latter should include (in sufficient detail) information of the topography, the nature of the soil, and the geologic formation of the ground, and should bring out those points of most importance relating to excessive cost of excavation, of construction, or of filling, inaccessibility of conduit, and difficulty of hauling materials; all the more difficult conduit sections *en route*, as also the choice of other shapes and of other routes; also notes should be taken which deal with any special conduit section such as siphons, aqueducts, flumes, sand-basins, weirs, etc. Before making a final choice, it may be important to ascertain (for different conditions) the economical velocity for a given grade or for different grades and a given design or different designs of conduit, and/or the *most economical conduit* section for a given energy loss, velocity, and grade. In the investigation of conduit location, the available slope or grade will depend upon the topographic and geologic conditions, the value of the coefficient of roughness usually being settled according to the alignment and material, etc., of the proposed class of conduit, but to satisfy these conditions it is necessary to determine the cross-section of the conduit.

For the ordinary conduit of constant section and uniform velocity of flow, let aa' and $b'b''$ in Fig. 13 represent the cross-sections normal to the direction of motion for a distance dL . Since the mass aa' and bb' moves uniformly, the external forces acting on it are in equilibrium. Let A be the area of cross-section, w the wetted perimeter of the section, then the quantity $R = A/w$ is termed the hydraulic mean depth of the section. Now, let V be the mean velocity of flow in the

conduit, and let S be the slope or fall of the conduit in feet per foot, making the ratio $b'c/ab'$. Then the external forces acting on aa' , $b'b''$, parallel to the direction of motion, can be described as :—

1. The component of the weight W of the mass in the direction of motion is that acting at its centre of gravity g' . Let $g = 62.4$ lbs. weight of water per cubic foot ; A = cross-section of conduit in square feet ; L = length of conduit in feet ; and S = fall in feet per foot. Then weight of the mass aa' , $b'b'' = gAdL$, and the component of the weight in the direction of motion is $gAdL$ multiplied by the cosine of the angle between Wg and ab' , that is,

$$gAdL \cos ab'c = gAdL(b'c/ab') = gASdL.$$

2. The pressure on aa' and $b'b''$ is equal and opposite since the sections are equal and similar.

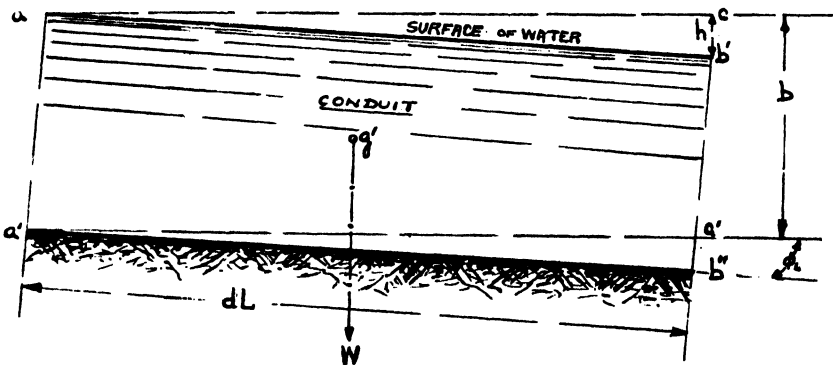


FIG. 13.—Showing section of gravity conduit.

3. The friction on the sides and bottom of the channel is proportional to the area $A dL$ of rubbing surface and to the function of velocity V' or friction per foot at a velocity V ; hence the friction is $-wdLV'$.

Equating the sum of the forces to zero, we have :—

$$gASdL - wLV' = 0 ;$$

but

$$V'/g = AS/w = RS,$$

$$V' = \eta g V^2 / 2g' = \eta V^2 / 2g' = RS,$$

which may be put in the form

$$V = \sqrt{(2g'/\eta)} \sqrt{RS} = c \sqrt{RS},$$

where c is a coefficient depending on the roughness and form of the channel. The flow or discharge is $Q = AV = Ac\sqrt{RS}$. The difficulty lies with a proper calculation or assumption of the coefficient c in any particular case for the flow in open channels.

$S = h/L$ = fall of water surface h , in any distance L , divided by that distance.
 $h = SL$; and $L = h/S$.

For Manning's formulæ (in which η has the same value as Kutter's formulæ) the value of c , as already explained, is $(1.4867/\eta)\sqrt[3]{R}$, then

$$V = \frac{1.4867}{\eta} \sqrt[3]{R} \times \sqrt{RS} = \frac{1.4867}{\eta} R^{1/6} \times R^{1/2} \times \sqrt{S} = \frac{1.4867}{\eta} \sqrt[3]{R^3} \sqrt{S}.$$

In terms of area (A) of section, depth of channel b , and the ratio of side slopes, the proportions of channels of maximum discharge for given area and side slopes are given in the following table :—

TABLE X.—SHOWING HYDRAULIC PROPERTIES FOR MAXIMUM DISCHARGE OF CHANNELS

Inclination of Sides to Horizontal.	Area of Section.	Bottom Width.	Top Width.	Ratio of Side Slopes.	Shape of Channel.
...	$1.571b^2$	...	$2b$	(Semicircle)	Semicircle
$60^\circ \quad 0'$	$1.732b^2$	$1.550b$	$2.310b$	3 : 5	Semi-hexagon
$90^\circ \quad 0'$	$2.000b^2$	$2.000b$	$2.000b$	0 : 1	Semi-square
$75^\circ \quad 58'$	$1.812b^2$	$1.562b$	$2.062b$	1 : 4	Prism
$63^\circ \quad 26'$	$1.736b^2$	$1.236b$	$2.236b$	1 : 2	"
$53^\circ \quad 8'$	$1.750b^2$	$1.b$	$2.500b$	3 : 4	"
$45^\circ \quad 0'$	$1.828b^2$	$0.828b$	$2.828b$	1 : 1	"
$38^\circ \quad 40'$	$1.952b^2$	$0.702b$	$2.202b$	1.25 : 1	"
$33^\circ \quad 42'$	$2.106b^2$	$0.606b$	$3.606b$	1.50 : 1	"
$29^\circ \quad 44'$	$2.282b^2$	$0.532b$	$4.032b$	1.75 : 1	"
$26^\circ \quad 34'$	$2.472b^2$	$0.472b$	$4.472b$	2 : 1	"
$23^\circ \quad 58'$	$2.674b^2$	$0.424b$	$4.924b$	2.25 : 1	"
$21^\circ \quad 48'$	$2.885b^2$	$0.385b$	$5.385b$	2.50 : 1	"
$19^\circ \quad 58'$	$3.104b^2$	$0.354b$	$5.854b$	2.75 : 1	"
$18^\circ \quad 26'$	$3.325b^2$	$0.325b$	$6.325b$	3 : 1	"

The velocity of water in a conduit is affected by the roughness of the wetted surface (sum of wetted sides) of the conduit and by the grade. The fixing of the grade determines, more or less, the velocity of water in the conduit and the area of cross-section. This grade may be such that the velocity will be low enough to permit the use of an earth type section (unlined), but generally the most economical conduit of this section (earth type) is the one lined with concrete. In fact, it is often the case that lining an earth section with concrete increases the discharge capacity and reduces the excavation costs and the seepage losses to such an extent that the cost of lining is more than offset by the sum total advantages gained. Furthermore, a saving in a few feet of grade, due to a better-designed conduit, resulting in a great reduction of the coefficient of roughness η , usually brings about economies in other directions (apart from less sectional

area, less material and labour, etc., for equal discharge capacity), such as relative reduction in flooded land, depending on the topography and type of development. This is accomplished by the more favourable location of the conduit intake, and can be made possible by taking full advantage of the better design and the higher value of η ; it might also decide the best and most economical forebay location.

In the case of nearly all water-power projects, the available stream flow (water-power supply forming the basis for power development) determines the capacity of the conduit. Usually the conduit (particularly long conduits) is designed for a constant flow, *plus* a percentage additional capacity. That is to say, a percentage is very often added to the *theoretically* determined cross-sectional area as found by the usual formulæ. Several important reasons exist for adding to the cross-sectional area of conduit as determined by formulæ, the most important being that of allowing for sufficient additional capacity to handle peak loads. After the available flow is decided upon, the most suitable shape of the different sections can be ascertained; then follows a determination of the sectional area or economical area of conduit; finally, the important factor of grade for the most suitable shape is considered. In arriving at the best design, this factor is usually allowed to vary—as a matter of fact, there is often no other course open, except that of keeping within the allowable cost or the limits of economical cost and best grade for the particular geologic and topographic and other conditions. As the most economical velocity will depend to some extent upon the price paid for energy, it may be more profitable to allow the highest velocity—assuming the price paid for energy is low, and vice versa. A point to keep in mind is that, usually, an ample design for all classes of conduit is comparatively less expensive than a curtailed one.

The better the hydraulic properties of a conduit channel, such as the hydraulic radius R and coefficient of roughness η , the greater will be the velocity of flow, or, for a river channel, the more rapid will be the run-off. There is a certain relation between bottom width and depth that gives a maximum hydraulic radius and a resulting maximum discharge capacity for a given conduit grade. In many cases this best relation is restricted largely by the geologic conditions and the irregular and rolling nature and cross-slope of the conduit route, and, because of this restriction, full advantage cannot always be taken of the best hydraulic properties. For a rectangular-shaped conduit, where the bottom width is twice the depth, the greatest velocity of a given cross-section is obtained; in other words, this proportion gives the maximum hydraulic radius. If relatively wide and shallow conduit must be used (by taking any advantage of

excavation on level or slightly sloping ground), then special care in operation and maintenance must be taken, because, with any drawing down of the head, the net area will suffer a greater proportionate reduction than in the case of a deeper and narrower conduit, but seepage loss will be reduced, as this loss is a function of depth of water, inclination of sides, and ratio of depth to bottom width. The form of conduit most favourable to flow is therefore seen to be one whose top width is equal to the two side slopes, whose top and bottom width together are equal to the wetted perimeter, and whose hydraulic mean radius is equal to one-half the depth.

A very common conduit channel is the trapezoidal section with sides of many slopes. If the longitudinal slope of the channel is given and also the mean velocity, obviously an infinite number of channels could be shown satisfying slope conditions. Let $abcd$

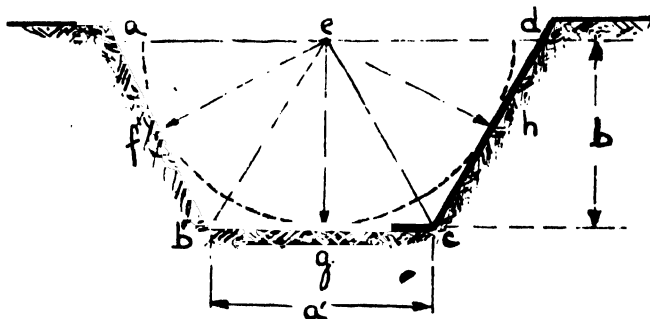


FIG. 14.—Showing the trapezoidal section.

be the channel, then from e , the centre of ad , drop perpendiculars ef , eg , and eh on the sides (see Fig. 14). Then

$$ab' = cd = a'$$

and

$$b'c = a', \text{ also } eg = b$$

but

$$ef = eh = b,$$

and

$$\text{Area} = A = aeb' + b'ec + ced = a'b + 1/2a'b$$

then wetted perimeter $= w' = 2a' + a'$,

also

$$a'b + 1/2a'b = (a' + 1/2a')b,$$

and ef , eg , and eh are equal.

A good practical limit of depth of water in the case of the earth-section type lies between 8 and 10 ft. For economy in excavation, conduits on level ground or with slight slope are sometimes made relatively wide and shallow, but on ground of steep slope they are sometimes made relatively deep and narrow. The minimum depth of a conduit will depend on the limits allowed by its location, topography and nature of ground, its design, type, and its relative

maintenance cost. For earth-section conduits the usual slope to give the sides (see Table XII.) is 1.5 to 1 (clayey loam), or, at the most, 2 to 1 (ordinary earth), but when the conduit is lined with concrete, or consists of masonry or good hardpan, the usual slope to give the sides is 1 to 1, although for concrete or masonry conduits any inclination may be given. Steep slopes of 0.5 to 1 usually stand fairly well in ordinary muck, depending on velocity of flow. Generally speaking, the steeper the sides, the less trouble from weed growth.

In a conduit, head loss (due to friction and grade) may be considered as corresponding to so many additional feet of conduit length; that is to say, a well-designed conduit of a given size and length can very well have a head loss appreciably less than a poorly-designed conduit of the same size but shorter. Loss of head in a conduit is due to its roughness, its design, its shape, its construction, its length, and the grade. The head loss increases with the square of the velocity. Intake loss of head is due to eddies, whirls, and to friction of the water and to the design and shape of the intake entrance. Loss of head at the entrance to penstocks is due to interior surface friction, to its design and shape, to the friction through racks, screens, valves, gates, and other obstructions to flow.

The safe *maximum* velocity, or speed of water in feet per second, can be taken as being approximately equal to 20 or even 30 ft. per sec. for concrete-lined sections; 10 to 16 ft. per sec. for rock-cut trimmed (unlined) sections; 6 to 8 ft. per sec. for the best clay loam; 3 to 4.5 ft. per sec. for ordinarily good loam; and 2 to 3 ft. per sec. for ordinary soil. In general, the use of unlined earth sections (earth-cut or ditch or canal) should be discouraged for the reason that the safe working velocity in some instances is barely high enough to prevent silt deposit and retard vegetable growth. Moreover, the ordinary unlined earth section of conduit is subject to serious loss by seepage. The higher the development head and the higher the price paid for power, the greater will be the value of a given quantity of water lost by seepage, wastage, evaporation, or to loss in any other way. Also, for the condition of high-priced energy, it is obvious that the value of head loss will be greater. Energy loss by seepage, although relatively much greater and more important, can, in one way, be likened to the "iron loss" in electric power transformers, in that it is a twenty-four hour a day loss. It is far more important than the power or energy loss in power-transmission conductors, which varies with the current load, in that, firstly, it is almost a constant energy loss, 8,760 hours in the year; secondly, the conduit cannot be built in duplicate, and thirdly, it cannot be operated in parallel like the transmission lines. Furthermore, for a full development output, the transmission line, electrical and

hydraulic plant, etc., may be dependent upon, and the power output limited by, these hydraulic losses.

The advantages of lining an earth, poor masonry, or rock-cut section with pitch, reinforced concrete,* or cement, etc., are well worthy of every consideration for all developments where water is scarce, seepage and evaporation losses high, and where water has been bought at great cost. The development *head* and conduit length are important factors in deciding the section to use; head loss is a function representing power loss. Some of the advantages for lined conduits now in the author's mind are:—

1. Lining of a conduit permits the use of a higher velocity than could be allowed in an unlined conduit.

2. The maintenance of a lined conduit is much less than that of one unlined.

3. The higher velocity permits the use of a smaller cross-section, with the consequent reduced expense of rock-cutting, excavation, etc.

4. A smaller conduit section is very often more practicable to construct for hillside conditions where a larger conduit may be unsafe or commercially impossible or only possible at excessive cost.

5. The lining of a conduit reduces the seepage loss to a negligible amount. In long earth, masonry, and rock-cut sections, where the materials are of poor and uncertain quality, seepage losses may be high.

6. With a lined conduit, the water will reach the forebay in less time than in the case of one not lined; also the water is more easily disposed of in case of a break in the conduit, and when shutting down for repairs.

7. The lining of a conduit permits the use of a section with highly desirable hydraulic properties, and, different from an earth section, maintains the sides and banks of the conduit so as to retain those qualities.

8. The lining of a conduit stops erosion of the banks by wave action, and delays the growth of weeds, brush, and moss, all of which interfere with the flow of water and increase deterioration.

9. Coating the concrete or cement lining with paint or pitch or varnish will tend to decrease seepage (where cracks occur) and increase the velocity of flow.

10. With an increase in depth of water in the conduit, the velocity and carrying capacity are increased. Without lining, an increase in depth would usually not be permissible.

11. For a given conduit and given flow, an increase in velocity results in a slight decrease in the seepage loss; lining the conduit will permit both advantages.

* To allow of expansion, it may be advisable to insert sections of asphaltum-felt about every 20 ft. of conduit length.

For a long length of earth conduit section (unlined) : if the earth is *not* underlined with clay or if the conduit does not have a surface of clay or impervious hardpan, then serious loss by seepage can be expected. Evaporation loss in conduits is relatively nil compared with the loss from seepage, and in actual practice is usually not accounted for. In certain of the less industrial countries, earth sections (unlined) are used because of financial difficulties. When used, a practical guide in the design is to allow for the following mentioned *loss* due to seepage per twenty-four hours : 0.5 to 1.5 cub. ft. per sq. ft. of conduit bed for medium clay loam ; 1.5 to 2 cub. ft. per sq. ft. of conduit bed for somewhat impervious soils ; about 1 cub. ft. per sq. ft. of conduit bed for medium soils ; and about 0.5 cub. ft. per sq. ft. of conduit bed for impervious clay soils. As already mentioned, seepage is a function of depth of water. Therefore, in order to care for a given discharge capacity, the width should be increased as the depth of flow is decreased. But, unfortunately, width is a factor in aggregating eddies, because the wider the conduit, the greater will be the number of eddies ; also high velocities and shallow depths aggregate eddies and consequent losses. In the matter of location, *width* is an important factor, but in the matter of design *depth* is an important function of loss by seepage. This loss is not strictly dependent on the quantity of water conducted through the conduit. A newly-made unlined earth section may give a much greater loss than the same conduit after several months' or years' usage, because the bottom and sides may by then have become coated with a thick film of impervious clay. Very often loss of water by seepage can be allowed in the early stages of a new water-power development, because of the usually favourable load conditions, *i.e.*, relatively low load factor, which, for the time being, favours this type of conduit.

It is usually required to find the dimensions and shape of a conduit for a given discharge and fixed grade. Also the problem sometimes met with is that of finding the dimensions for various assumed or possible grades, and at other times it is required to know the most economical dimensions of a conduit. For the former case, several assumed dimensions are taken to satisfy equation Q/A for *mean* velocity V , and the coefficient of roughness is taken from a value known in practice to be reasonably reliable for the particular type and kind of conduit decided upon (see Fig. 15). Thus, for fixed values of Q , η , and S , let

$$Q = 200 \text{ cusecs, } S = 0.0013, \text{ and } \eta = 0.013,$$

and let us assume five different dimensions, such as :—

$A = 60, 50, 40, 30, \text{ and } 20 \text{ sq. ft. (respectively), then respective mean velocity}$
 $V = Q/A = 200/60 = 3.33 ; 200/50 = 4.0 ; 200/40 = 5.0 ; 200/30 = 6.66 ;$
 and $200/20 = 10 \text{ ft. per sec.}$

Tabulating for η , V , and S with Q , η , and S *fixed*, determine R from Manning formula, or take from the logarithmic diagram, Fig. 15, we obtain :—

A	Q	η	S	V	R
60	200	0.013	0.0013	3.33	0.74 ft.
50	200	0.013	0.0013	4.00	1.00 „
40	200	0.013	0.0013	5.00	1.4 „
30	200	0.013	0.0013	6.66	1.93 „
20	200	0.013	0.0013	10.00	4.0 „

Let us also assume a shape so that $R = b/2$, wherein b = depth. And let b equal 6, 5, 4, 3, and 2 ft. in depth of water respectively ; then, for these different assumed depths of water we have :—

$$R' = 2/2 = 1.0 ; 3/2 = 1.5 ; 4/2 = 2.0 ; 5/2 = 2.5 ; \\ \text{and } 6/2 = 3.0 \text{ ft. hydraulic radius (respectively).}$$

Then, checking these assumed values of R' with calculated values of R , the relation would be :—

$$R = 4.0 ; 1.93 ; 1.4 ; 1.0 ; \text{ and } 0.74 \text{ ft.} \\ R' = 3.0 ; 2.50 ; 2.0 ; 1.5 ; \text{ and } 1.00 \text{ ft.}$$

In terms of wetted perimeter (length of wetted sides), we may write $R = A/2b + a$; wherein a = width of conduit of this shape.

In this case we need only move $R' = 2$ ft. under $R = 1.93$ ft., which gives a conduit dimension of 8 ft. wide by 4 ft. deep, or $A' = 2b^2 = 2 \times (4 \times 4) = 32$ sq. ft. ; that is, in terms of wetted perimeter $R = A/2b + a = 32/(2 \times 4 + 8) = 2$ ft. hydraulic radius.

For the assumed depths of water we have a corresponding cross-section of—

$$A' = 2b^2 = \begin{array}{ll} 2 \times 2^2 = 8.0 \text{ sq. ft. cross-section,} \\ 2 \times 3^2 = 18.0 & \text{,,} \text{,,} \\ 2 \times 4^2 = 32.0 & \text{,,} \text{,,} \\ 2 \times 5^2 = 50.0 & \text{,,} \text{,,} \\ 2 \times 6^2 = 72.0 & \text{,,} \text{,,} \end{array}$$

and corresponding mean velocity—

$$V' = \frac{Q'}{A'} = \begin{array}{ll} 200/72 = 2.78 \text{ ft. per sec.} \\ 200/50 = 4.00 \text{ ,,} & \text{,,} \\ 200/32 = 6.25 \text{ ,,} & \text{,,} \\ 200/18 = 10.11 \text{ ,,} & \text{,,} \\ 200/8 = 25.00 \text{ ,,} & \text{,,} \end{array}$$

Tabulating these various values of A' , V' , and R' for *fixed* flow Q' and for *fixed* value of η , we obtain the following values of S' —

Q'	A'	V'	R'	η	S
200	72	2.78	3.0	0.013	0.00014
200	50	4.00	2.5	0.013	0.00030
200	32	6.25	2.0	0.013	0.00096
200	18	10.11	1.5	0.013	0.00360
200	8	25.00	1.0	0.013	0.00450

Only for the purpose of calculation let cross-section (although small) equal 30 sq. ft., with a slope of 1.3 ft. per 1,000 ft. and hydraulic radius of 1.924 ft. Therefore, for *fixed* values of A , η , and R we have—

A	R	η	Q	V	S	h (per 1,000 ft.)
30	1.924	0.013	50	1.66	0.00009	0.09 ft.
30	1.924	0.013	100	3.33	0.00035	0.35 „
30	1.924	0.013	150	5.00	0.00070	0.70 „
30	1.924	0.013	200	6.66	0.00130	1.30 „
30	1.924	0.013	250	8.33	0.00300	3.00 „

In terms of grade, *power loss* in the conduit can be expressed—

$$p = QLS e'; \text{ but } LS = h \text{ and } S = h/L;$$

therefore

$$p = Qhe';$$

wherein Q = discharge in cusecs.

L = length of conduit in feet.

S = grade or slope in feet per foot of conduit.

e' = efficiency of water (from head-works intake to turbine shaft or at switchboard),

multiplied by 0.1134 for h.p.

multiplied by 0.1000 for e.h.p.

multiplied by .0846 for kW.

Example.—Take 10 miles (52,800 ft.) of conduit length (varnished, and having a good, smooth concrete section) constructed on a grade

so that 1,000 ft. dissipates 1.3 ft.-head, the discharge is 200 cusecs, and let total efficiency be 95 per cent. ; then, power loss in horse-power value is—

$$p = QLSe' = 200 \times (52,800 \times 0.0013) \times (0.95 \times 0.1134) = 1,400 \text{ h.p.},$$

or $p = Qhe' = 200 \times 68.64 \times 0.10773 = 1,400 \text{ h.p. (due to grade).}$

Also, by the common formula—

$$p = \frac{Qhe'}{8.82} = \frac{200 \times 68.64 \times 0.95}{8.82} = 1,400 \text{ h.p.}$$

Assuming a development head of 800 ft., and a loss by seepage * in the conduit of 0.0175 cusec (per 1,000 ft. per day of twenty-four hours), then total loss is $52.8 \times 0.0175 = 0.924$ cusec ; making an energy loss of—

$$0.924 \times 800 \times 0.10773 = 79.63 \text{ h.p. per twenty-four hours,}$$

or $365 \times 79.63 = 29,066 \text{ h.p. hours per year,}$

and $29,066/8,760 = 3.32 \text{ h.p. (loss due to seepage).}$

For a concrete-lined conduit, coated with a film of pitch or varnish,† this loss by seepage is given as the absolute maximum value, hence it can be neglected in the calculations in view of the fact that the total developed power is—

$$QHe' = 200 \times 800 \times 0.10773 = 17,237 \text{ h.p.}$$

For the same unlined earth section, the seepage loss would be about 1,900 h.p. per twenty-four hours = 693,500 h.p. hours per year, or possibly much greater ; but the same shape could not be used, and for equal Q the losses would be much greater.

Tabulating results for fixed discharge capacity (200 cusecs), different sections in function of depth, conduit cost per 1,000 ft. linear, annual interest at 6 per cent., and assumed value of power per horse-power year at £3, £4, and £5 respectively, we may draw curves showing *the minimum values of the sum of annual cost of conduit plus annual value of power loss*. For these conditions, the *most economical section* is seen to come between the conduit depth of 4 ft. (32 sq. ft.) and 5 ft. (50 sq. ft.) ; see g and g' in following table.

* 1 cusec for twenty-four hours equals 86,400 cub. ft., hence loss of 1 ft. in depth over wetted area (length of wetted sides) in one day of twenty-four hours can be expressed as $q = (W' \times 1,000)/86,400$ (per 1,000 ft. of conduit length) ; W' = wetted perimeter in feet ; q = loss in cusecs per 1,000 ft. length of conduit.

† For ordinary concrete section, the value of coefficient $\eta = 0.015$ to 0.017, but for very smooth surface such as varnished or pitch-coated concrete section, $\eta = 0.013$, or better.

CONCRETE CONDUIT (VARNISHED OR TREATED WITH COAT OF PITCH) CARRYING 200 CUSECS ($\eta = 0.013$)

<i>a</i>	<i>b'</i>	<i>c</i>	<i>d</i>	<i>e</i>	A		B		C	
					<i>f</i>	<i>g</i>	<i>f'</i>	<i>g'</i>	<i>f''</i>	<i>g''</i>
2	1,350	81	4.5	91.8	275.4	356.4	367.2	448.2	459.0	540.0
3	1,850	111	3.6	73.45	220.3	331.3	223.8	334.8	367.25	478.25
4	2,500	150	0.96	19.6	58.8	208.8	78.4	228.4	98.00	248.0
5	3,000	180	0.30	6.12	18.36	198.4	24.5	204.5	30.60	210.6
6	3,750	225	0.14	2.86	8.58	233.6	11.45	236.45	14.3	239.3

a = conduit water depth (b)* in feet ; wherein $A' = 2b^2$, and $R' = b/2$.

TABLE XI.—SHOWING MOST FAVOURABLE CONDUIT CROSS-SECTION

Character of Material.	Proportional Areas.	Bottom Width <i>a</i> in Terms of Vertical Depth <i>b</i> .	Conduit Side-Slope Ratios.
Concrete and masonry {	1.0	$2b$	0 to 1.0
	0.906	$1.562b$	0.25 „ 1.0
	0.868	$1.236b$	0.5 „ 1.0
Muck — — — } Hardpan — — — } Clay loam — — — } Ordinary earth — — — }	0.914	$0.828b$	1.0 „ 1.0
	1.053	$0.606b$	1.5 „ 1.0
	1.236	$0.472b$	2.0 „ 1.0

TABLE XII.—GIVING CHANNEL RELATIONS IN TERMS OF $\sqrt{A} = \sqrt{\text{AREA}}$

Conduit Side-Slope Ratios.	Bottom Width.	Depth.	Width of Water Surface.	Hydraulic Mean Radius.	Wetted Perimeter.
0 to 1.0	$1.414 \sqrt{A}$	$.707 \sqrt{A}$	$1.414 \sqrt{A}$	$.353 \sqrt{A}$	$2.828 \sqrt{A}$
1.0 „ 1.0	$0.613 \sqrt{A}$	$.74 \sqrt{A}$	$2.093 \sqrt{A}$	$.369 \sqrt{A}$	$2.705 \sqrt{A}$
1.5 „ 1.0	$0.418 \sqrt{A}$	$.689 \sqrt{A}$	$2.485 \sqrt{A}$	$.344 \sqrt{A}$	$2.904 \sqrt{A}$
2.0 „ 1.0	$0.300 \sqrt{A}$	$.636 \sqrt{A}$	$2.844 \sqrt{A}$	$.318 \sqrt{A}$	$3.144 \sqrt{A}$

* In all cases the depth referred to means actual depth of water, the conduit area itself (for vertical section) is in the ratio of approximately 6 to 10, *i.e.*, bottom width in terms of depth of conduit from top (not water surface).

b' = estimated cost (in £) per 1,000 ft. length of conduit.

c = annual interest on the cost (in £) taken at 6 per cent.

d = loss of head per 1,000 ft. of conduit length.

e = power loss in horse-power per 1,000 ft. length of conduit.

$f f' f''$ = annual value of power loss in horse-power year taken at £3, £4, and £5 respectively.

$g g' g''$ = annual value of power loss in horse-power year, *plus* interest = $f + c$; $f' + c$; and $f'' + c$.

Thus *bottom width* in terms of *depth* as given in Table XI., for side slope of 0 - 1.0 (vertical) = $1.414/.707 = 2.00b$,

$$\text{for } 1.0 \text{ to } 1.0 = .613/.74 = 0.828b,$$

$$\text{for } 1.5 \text{ ,, } 1.0 = .418/.689 = 0.606b,$$

$$\text{and for } 2.0 \text{ ,, } 1.0 = .300/.636 = 0.472b.$$

For a concrete or masonry conduit with vertical side slopes (0 - 1.0) and with the following properties :—

$$Q = 200 \text{ cusecs ; } A = 50 \text{ sq. ft. ; } R = 2.5 \text{ ft. ; } W' = 20 \text{ ft. ;}$$

$$b = 5 \text{ ft. ; and } a = 10 \text{ ft.,}$$

hence, by Table XII., we have for $\sqrt{A} = \sqrt{50} = 7.07$;

$$\text{bottom width, } a = 1.414 \times 7.07 = 10 \text{ ft. ;}$$

$$\text{depth, } b = .707 \times 7.07 = 5 \text{ ft. ;}$$

$$\text{width of water surface, } a = 1.414 \times 7.07 = 10 \text{ ft. ;}$$

$$\text{hydraulic mean radius, } R = .353 \times 7.07 = 2.5 \text{ ft. ;}$$

$$\text{and wetted perimeter, } W' = 2.828 \times 7.07 = 20 \text{ ft.}$$

The usual assumed and accepted conditions met with in practice for the design of water-power conduits are :—

1. Available quantity Q of water for the power development is *fixed*.

2. Roughness factor η (dependent upon the channel conditions) is *fixed*.

3. Length of conduit is *fixed*.

4. Proper slope to give the sides of the channel, which depends upon the stability of material ; usually *fixed*.

5. Efficiency of water Q to point of delivery, multiplied by a coefficient, is usually *fixed*.

6. Safe velocity of flow is dependent upon the stability of the material ; usually *fixed* for best speed of water.

7. Surface slope or grade of conduit is *usually the variable* factor, and is dependent generally upon natural conditions of the surveyed and accepted route.

The common problem is to decide upon a shape of conduit, and then determine the economical area and slope for this shape. Perhaps other sizes, shapes, and slopes along the route of a given conduit installation will be found desirable, due to the change in topography, etc. A determination should, of course, be made for the best possible shapes which would produce the most advantageous cross-sections. The quantity Q should be greatest for the least friction, *i.e.*, when the wetted perimeter ω' is *least* (see tabulation of examples given below) for a given area A of conduit, that is, when $R = A/\omega' = \text{maximum}$, under the conditions offered. As the value of η is fixed (for designing purposes), then, because of the natural conditions *en route*, the slope or grade may have to be varied, and therefore any change in the power loss would be due to a change of

the slope or grade. Assuming that the shape and slope for one type of conduit section have been determined, then, for certain parts of the conduit route the question might arise whether it would not be better, cheaper, or only possible (due to the sloping nature of the ground, etc.) to make some change either in the shape, in the slope, or in the size of conduit, so as to obtain the best and most advantageous cross-section and/or maximum possible increase in Q , and/or most economical decrease in power loss due to slope of conduit, etc. In practice, the discharge quantity Q will vary with the load factor, but for designing purposes it is always taken as constant, because, among other reasons, sufficient water must be situated where it can be drawn upon at a moment's notice, *i.e.*, in the forebay or regulating basin. Therefore, any change in the slope or grade may be offset by a change in the design, either in the area A , in R , or in both, *i.e.*, either the size or the shape of conduit may be varied to keep Q fixed at or near its maximum.

The logarithmic diagram shown in Fig. 15 will be found useful for estimating the flow of water in open conduits. Where greater accuracy is required, the size of the diagram should be increased. Usually, charts and diagrams of this nature are too small when put into book page-form and not made to fold several times. The page size given here will, however, be found sufficiently close enough for preliminary designs; in fact, results obtained from Fig. 15 may be more accurate than base information involved in the reconnaissance survey and general knowledge of the stream, sites, etc., and/or the estimated characteristics; and/or the type of waterway used or decided upon.

Given any of the three variables R , S , η , and V , any one of these variables can be found from Fig. 15. That is to say—

(a) *Given R , η , and S* ; follow the vertical value of R to the intersection of η , then horizontally to S ; at this intersection the value V is obtained as read at the top of the chart.

(b) *Given S and V , and assume η* ; follow the vertical value of V to the intersection of S , then move horizontally to the assumed value of η , when the value of R is found at the bottom of chart.

(c) *Given η , R , and V* ; follow the vertical line for value R to the intersection of value η , then from value V follow vertically to the slope values S until this said value intersects the value.

(d) *Given S , R , and V* ; to find η follow V vertically to S , then from value R move vertically until the horizontal value S meets the vertical line V , when η is found at the point of intersection.

To illustrate the use of Fig. 15, take the following example :—

Example.—What is the mean velocity of water in feet per second, and the discharge in cubic feet (ft.³/sec.) from an open conduit (earth

channel unlined) having a hydraulic mean depth of 6.5 ft. and a slope of 0.0001 ft. per ft.; η is taken at 0.025? The cross-sectional area of the channel is 417.5 sq. ft.

From Table XII. the hydraulic mean depth is

$$R = 0.318 \sqrt{A} = 0.318 \sqrt{417.5} = 0.318 \times 20.44 = 6.5 \text{ ft.}$$

In this example it is required to find the mean velocity V and the discharge Q . To find the velocity V , follow the vertical value $R = 6.5$ ft. (right side of chart) to the intersection of $\eta = 0.025$, then horizontally to the diagonal line for $S = 0.0001$; at this intersection read vertically, we find mean velocity $V = 2.07$ ft. per sec.

Hence, $Q = AV = 417.5 \times 2.07 = 864$ cusecs or ft.³/sec.

Calculating the velocity from Manning's formulæ, we get—

$$\frac{1.4856}{\eta} \sqrt{R^3} \sqrt{S} = \frac{1.4856}{0.025} = 59.43; \quad \sqrt[3]{(6.5)^2} = 3.483;$$

see values in Fig. 12 (p. 89); or from table on p. 84, for values of $\sqrt[3]{R^2}$; $\sqrt{0.0001} = 0.01$; also see values in Fig. 12;

then $V = 59.42 \times 3.484 \times 0.01 = 2.07$ ft. per sec.
and $Q = 417.5 \times 2.07 = 864$ cusecs.

Now take the case of an earth-section conduit where (due to topography and nature of the ground) the depth of water in the conduit is expected to vary; determine the sizes (dimensions) and the slopes for a *given* velocity, shape of conduit, and roughness factor. For example: A conduit (somewhat irregular and rough) is required to carry 200 cusecs at a mean velocity of about 2.5 ft. per sec. Throughout the channel the slope has practically 1 to 1 side slopes. The depth of water in the conduit might vary from 4 ft. in some cases to 6 ft. in others. Determine the slopes and the dimensions of the channels which might be used. The value of coefficient of roughness is taken at 0.025.

$$\text{Area of waterway } A = Q/V = 200/2.5 = 80 \text{ sq. ft.};$$

then, for varying depths of channel, we have in terms of depth (b):—

	Depth (b) in Feet.	Base (a) in Feet.
Base = $a = (A/b) - b$	4.0	16.0
	4.5	13.27
	5.0	11.00
	5.5	9.04
	6.0	7.33

but $A = (a + b)b = (16 + 4)4 = 80$ sq. ft.

$$(11 + 5)5 = 80 \text{ sq. ft.}$$

$$(7.33 + 6)6 = 80 \text{ sq. ft.}$$

also $A = ab + b^2 = (4 \times 16) + (4 \times 4) = 80$ sq. ft.

$$(5 \times 11) + (5 \times 5) = 80 \text{ sq. ft.}$$

$$(6 \times 7.33) + (6 \times 6) = 80 \text{ sq. ft.}$$

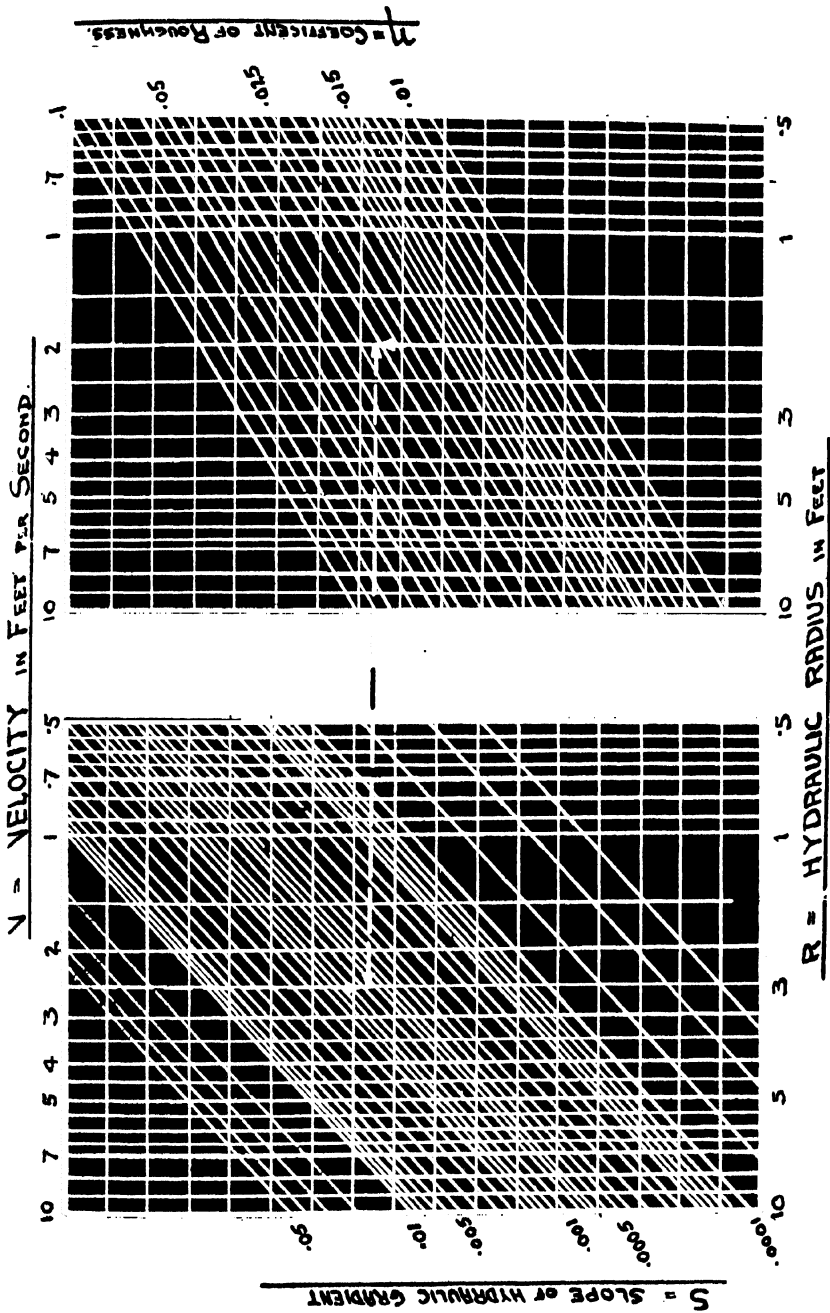


FIG. 15.—Diagram for flow of water in open conduits.

For side slopes of 1 to 1, wetted perimeter is

$$\begin{aligned} w' &= a + (2.828b) = 16 + (4.0 \times 2.828) = 27.3 \text{ ft.} \\ &13.27 + (4.5 \times 2.828) = 26.0 \text{ ft.} \\ &11 + (5.0 \times 2.828) = 25.14 \text{ ft.} \\ &9.04 + (5.5 \times 2.828) = 24.59 \text{ ft.} \\ &7.33 + (6.0 \times 2.828) = 24.3 \text{ ft.} \end{aligned}$$

and hydraulic mean depth is

$$\begin{aligned} R &= \frac{ab + b^2}{a + 2.828b} = A/w' = 80/27.3 = 2.93 \text{ ft.} \\ &80/26.0 = 3.08 \text{ ft.} \\ &80/25.14 = 3.18 \text{ ft.} \\ &80/24.59 = 3.25 \text{ ft.} \\ &80/24.3 = 3.29 \text{ ft.} \end{aligned}$$

Then, for 80 sq. ft. sectional area (fixed), we have, from Table XII.—

$$\begin{aligned} a &= 0.613 \sqrt{A} = 0.613 \times 8.92 = 5.47 \text{ ft.} \\ b &= 0.740 \sqrt{A} = 0.740 \times 8.92 = 6.60 \text{ ft.} \\ w' &= 2.705 \sqrt{A} = 2.705 \times 8.92 = 24.2 \text{ ft.} \\ R &= 0.369 \sqrt{A} = 0.369 \times 8.92 = 3.29 \text{ ft.} \end{aligned}$$

or, for one condition of channel, we have—

$$\begin{aligned} a &= 0.828b = 0.828 \times 6.6 = 5.47 \text{ ft.} \\ w' &= a + (2.828b) = 5.47 + (2.828 \times 6.6) = 24.2 \text{ ft.} \\ R &= 0.5b = 0.5 \times 6.6 = 3.3 \text{ ft.} \end{aligned}$$

From Fig. 11 (using curve B, for $\eta = 0.025$) the slopes for respective values of R, η , V will be about—

R	V	η	S
2.82	2.5	0.025	0.00042
3.08	2.5	0.025	0.00041
3.18	2.5	0.025	0.00040
3.25	2.5	0.025	0.00039
3.29	2.5	0.025	0.00038

Mean velocity from Manning's formulæ for one condition of channel, in which R = 3.25 ft. and S = 0.00039, is

$$\begin{aligned} V &= \frac{1.4858}{\eta} \sqrt{R^3} \sqrt{S} \\ \frac{1.4858}{0.025} &= 59.43 ; \sqrt{(3.25)^3} = 2.194 ; \sqrt{0.00039} = 0.0198 \text{ (see p. 84).} \\ &= 59.43 \times 2.194 \times 0.0198 = 2.58 \text{ ft. per sec.} \end{aligned}$$

The length of any open conduit will depend upon the topography and other local conditions, but let us assume that this conduit is

used with a hydro-electric plant working under a head of 700 ft. Also assume that, because of leakage and head loss in the conduit, the power output is unduly reduced by 2 per cent. Let hydro-electric *plant* efficiency be 85 per cent., that is

$$kW. = 700 \times 200 \times 0.0846 \times 0.85 = 10,000 \text{ kW.}$$

For 83 per cent. efficiency about 200 kW. is lost ; thus, a difference of 2 per cent. efficiency represents 200 kW., or 100 per cent. load factor,

$$200 \times 8,760 = 1,752,000 \text{ kW.-hours per year,}$$

which, at 0.5 pence per kW.-hour, is £3,650 ; capitalised at 10 per cent. is £36,500 as a value of 2 per cent. This amount may be as great or even greater than the total cost of the conduit, and its importance (especially if it be a long conduit) may be greater than that of any other waterway forming the hydraulic end of the complete hydro-electric development in that it may constitute what might be called the "bottle-neck" of the system. Under ordinary conditions, water and head loss in the conduit is as important as the same losses occurring in any other part of the hydro-electric system, but under full load and/or peak load conditions these losses may be of far greater value in several different ways.

Another point, as regards loss of power, is the soundness of the material through which the channel is cut. The writer recalls a case of design which was based upon the assumption of rock (as decided upon by field notes taken of material from the bore-holes along the surveyed route) forming 50 per cent. of the excavation material. Actually the surface material was soil and the subsurface material consisted of rock of insufficient thickness and with open joints and fissures ; these conditions were unknown at the time the design was made. The underlying material, which came above the bottom of the waterway, consisted of gravel mixed with loam, and the material being porous and the rock of very poor quality and unsound, a concrete lining (not originally allowed for) had to be provided for the whole length of the canal to avoid great loss of water and power. The introduction of a concrete lining permitted an increase in the values of Q and V .

The following table may be found useful in connection with $\sqrt{S}$ values for the hydraulic gradients used in water-power conduits :—

TABLE XIII.—TABLE OF CHANNEL GRADIENTS GIVEN TO WATER-POWER CONDUITS. THE VALUE $\sqrt{S}$ TO BE USED WITH MANNING'S FORMULÆ.

Slope (in Unity).	In Unity.	Feet per 1,000 Ft.	$\sqrt{S}$.
3,000	0.00034	0.34000	0.018439
2,778	0.00036	0.36000	0.018974
2,631	0.00038	0.38000	0.019494
2,500	0.00030	0.30000	0.020000
2,380	0.00032	0.32000	0.020494
2,273	0.00024	0.24000	0.020976
2,174	0.00026	0.26000	0.021448
2,083	0.00028	0.28000	0.021909
2,000	0.00020	0.20000	0.021360
1,923	0.00052	0.52000	0.022804
1,851	0.00054	0.54000	0.023238
1,786	0.00056	0.56000	0.023664
1,700	0.00058	0.58000	0.024083
1,666	0.00060	0.60000	0.024495
1,612	0.00062	0.62000	0.024899
1,544	0.00064	0.64000	0.025291
1,501	0.00066	0.66000	0.025691
1,470	0.00068	0.68000	0.026077
1,430	0.00070	0.70000	0.026458
1,489	0.00072	0.72000	0.026833
1,351	0.00074	0.74000	0.027203
1,316	0.00076	0.76000	0.027568
1,282	0.00078	0.78000	0.027929
1,250	0.00080	0.80000	0.028284
1,220	0.00082	0.82000	0.028636
1,190	0.00084	0.84000	0.028983
1,162	0.00086	0.86000	0.029326
1,136	0.00088	0.88000	0.029665
1,111	0.00090	0.90000	0.029833
1,089	0.00092	0.92000	0.030332
1,064	0.00094	0.94000	0.030658
1,041	0.00096	0.96000	0.030984
1,002	0.00098	0.98000	0.031305
1,000	0.00100	1.00000	0.031623
833	0.00120	1.20000	0.034641
714	0.00140	1.40000	0.037417
625	0.00160	1.60000	0.040000
556	0.00180	1.80000	0.042426
500	0.00200	2.00000	0.044721
400	0.00250	2.50000	0.050000

Example.—A conduit which has a slope of 1 in 2,500, falls 1 ft. in 2,500 ft. ; the fall is 0.00030 ft. per ft. of conduit,

$$\text{for } V = \frac{1.4868}{\eta} \sqrt[3]{R^2} \sqrt{S}; \quad (\sqrt{S} = 0.020000),$$

$$S = h/L = 1/3,333 = 0.00030 \text{ ft. per ft.,}$$

$$h = LS = 3,333 \times 0.00030 = 1 \text{ ft.}$$

Example.—

Let $Q = 200$ cusecs ; $V = 3.0$ ft. per sec. for *earth section* ; $\eta = 0.02$;
also, $Q = 200$ cusecs ; $V = 8.0$ ft per sec. for *concrete section* ; $\eta = 0.013$.
(Let side slopes of conduit in each case be 1.5 to 1.0.)

(A) *Earth Section* :—

$$A = Q/V = 200/3 = 66.6 \text{ sq. ft., making } \sqrt{66.6} = 8.144,$$

and, from Table XII. (p. 105)—

Depth of water	(b) = .689 × 8.144 = 5.611 ft.
Base of channel	(a) = .418 × 8.144 = 3.40 „
Wetted perimeter	(w') = 2.904 × 8.144 = 23.65 „
Hydraulic mean radius (R)	= .344 × 8.144 = 2.80 „

or, from formulæ for side slopes of 1.5 to 1.0 :—

$$A = ab + 1.5b^2 = (5.611 \times 3.4) + 1.5(5.611)^2 = 66.5 \text{ ft.}$$

$$w' = a + 3.61b = 3.4 + 3.61 \times 5.611 = 23.65 \text{ ft.}$$

$$R = \frac{ab + 1.5b^2}{a + 3.61b} = \frac{66.5}{23.65} = 2.8 \text{ ft.}$$

From Figs. 11 and 15 it is seen that for an earth section a channel which has $R = 2.8$ ft. must have a surface slope of about 0.0004 to give a velocity of 3 ft. per sec.

(B) *Concrete Section* :—

$$A = Q/V = 200/8 = 25 \text{ sq. ft., making } \sqrt{25} = 5.0,$$

therefore—

Depth of water	(b) = .689 × 5 = 3.445 ft.
Base of channel	(a) = .418 × 5 = 2.09 „
Wetted perimeter	(w') = 2.904 × 5 = 14.52 „
Hydraulic mean radius (R)	= .344 × 5 = 1.72 „

From Figs. 11 and 15 it is seen that a concrete channel section which has $R = 1.72$ ft. must have a slope of about 0.0025 to give a velocity of 8 ft. per sec.

In the case of the former (earth section) conduit, the *material* decides and fixes its best shape ; but not so (in this case) for the latter (concrete section) conduit. We shall, for the latter, take the most advantageous cross-section (vertical side slopes), leaving Q , V , A , and η *fixed*, as before. Thus—

(C) *Concrete Section* :—

$$\begin{aligned}
 \text{Depth of water} & (b) = .707 \times 5 = 3.535 \text{ ft.} \\
 \text{Base of channel} & (a) = 1.414 \times 5 = 7.07 \text{ ,,} \\
 \text{Wetted perimeter} & (\omega') = 2.828 \times 5 = 14.14 \text{ ,,} \\
 \text{Hydraulic mean radius} & (R) = .353 \times 5 = 1.765 \text{ ,,}
 \end{aligned}$$

Or, from formula for vertical side slopes :—

$$\begin{aligned}
 A &= ab = 2b^2 = 2(3.535)^2 = 25.0 \text{ sq. ft.} \\
 \omega' &= a + 2b = 7.07 + (2 \times 3.535) = 14.14 \text{ ft.} \\
 R &= \frac{ab}{a + 2b} = \frac{b}{2} = \frac{3.535}{2} = 1.767 \text{ ft.}
 \end{aligned}$$

It is therefore seen that when the wetted perimeter ω' is *least* for a given area A of conduit (*i.e.*, when $R = A/\omega' = \text{maximum}$, or, when the friction is least), the flow will be greatest. By tabulating, for the above examples we may write :—

EXAMPLE. Material - - - - -	(A) Earth Section.	(B) Concrete- Lined.	(C) Concrete- Lined.
Side-slopes - - - - -	1.5 to 1.0	1.5 to 1.0	Vertical
Q in cusecs - - - - -	200	200	200
V in feet per second - - - - -	3.0	8.0	8.0
η (good) - - - - -	0.02	0.013	0.013
A in square feet - - - - -	66.6	25.0	25.0
Base of conduit channel in feet - - - - -	3.4	2.09	7.07
Depth of water in feet - - - - -	5.611	3.445	3.535
Width of water at top in feet - - - - -	20.238	12.425	7.07
Wetted perimeter in feet - - - - -	23.65	14.52	14.14
Hydraulic mean radius in feet - - - - -	2.8	1.72	1.765
Slope in feet per foot - - - - -	0.0004	0.00225	0.0025
Q loss per day per 1,000 ft. due to seepage (approximate)	0.55	0.017	0.016
Horse-power loss (per 1,000 ft. of conduit length) due to slope (100 per cent. water efficiency)	9.047	51.0	56.7
Total gross horse-power per 100 ft.- head	2,268	2,268	2,268

The final problem in all cases should reduce itself, as in the case of every class of energy conduit, to the application of Kelvin's law, which is used for the bare overhead line conductor of the electric circuit. Similar to *pressure pipe lines*, the best way to arrive at the most economical cross-section is to make detailed estimates for several cross-sections (most advantageous cross-sections) and determine from an analysis of these estimates the best size of conduit.

Taken as a general rule, the writer recommends that about 15 per cent. be added to the calculated cross-section as obtained by the usually accepted formulæ. Thus, in the case of the 32 sq. ft. cross-section mentioned above, a size giving $32 \times 1.15 = 37$ sq. ft., would be the more advantageous cross-section and the better of the two conduits (see p. 102, also table on p. 105).

In this treatment of the hydraulic conduit, with particular reference to the gravity, non-pressure or *open type conduit*, such matters as siphons, aqueducts, sand-basins, weirs, gates, etc., in connection therewith have been purposely omitted from the text.

At every conduit-intake there should be installed some device for taking care of debris. Preferably, double screens or strainers (sometimes referred to as racks) should be used; one with 2-in. openings between the bars, and one finer. This also applies to the forebay end of the waterway, near the pipe intake, *i.e.*, nearest to the turbine. This latter should not have more than 0.75-in. openings. The size of the bars should be about 2 in. by $\frac{5}{8}$ in. When an open water-conduit runs through forest, it is usually found necessary to have a strainer so arranged that leaves and branches from trees, etc., are collected; such a strainer can be operated by a hand rack or by a worm gear. Sluice gates are generally used in conduits. These gates are raised and lowered by means of racks and pinions, the pinions being actuated through worm-gear which may be driven by hand or by electric motor.

Conduit Installations

1. *High Efficiency Conduit.*—The river ordinarily is characterised neither by floods nor by droughts in the dry season, and an average minimum of from 500 cusecs to 600 cusecs is continuously available. By means of a conduit 3.068 miles long from its intake to the discharge flume, there is secured an effective head of 470 ft. Of the entire length of conduit, about 25 per cent. is open conduit (earth cut, concrete-lined) and about 68 per cent. is tunnel (lined). The remaining 7 per cent. constitutes the discharge flume and tail-race. The open conduit is generally about 8.5 ft. wide at the bottom, with slanting reinforced concrete side walls to the height of 9.5 ft. Sections of asphaltum-felt are inserted every 18 ft. to prevent expansion. The tunnel section is also lined with concrete 6 to 8 in. thick, and is varnished to a thickness of about $\frac{1}{2}$ in. to reduce friction. Two sand drainages are situated along the open conduit with large silt pits dug down 4 ft. below the conduit bed.

2. The intake to this conduit is at the south end of the dam, a sluice-way for debris intervening; the face of the intake is at an

angle of about 30° with the line of the dam, producing a cross-current towards the sluice-way. The intake is 100 ft. over all, and is provided with five 16-ft. openings; the openings are provided with proper racks and screens to exclude debris; the depth of water is 14.5 ft., to obtain which rock excavation was necessary. The entrance velocity is low, being about 2.5 ft. per sec. The top of the structure is 6 ft. above working level; thus these head-works are 2 ft. above extreme flood level, the dam being designed for a maximum overtop of 4 ft. The conduit is 1,900 ft. in length; from the intake it is entirely in excavation. The earth is really gravel mixed with loam, and the material being porous, a concrete lining is provided for the whole length of the channel. The section in the different material

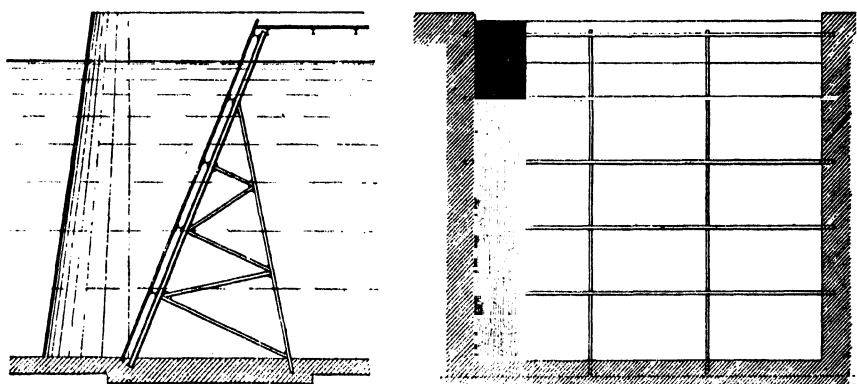


FIG. 16.—Showing type of screen or rack used at the intake to conduit and at the forebay end of the conduit.

is so adjusted as to obtain an area of cross-section of at least 525 sq. ft., giving a mean velocity in the conduit, under working conditions, of from 3 to 4 ft. per sec.

3. This conduit is 2,600 ft. long; it has a cross-sectional area of 500 sq. ft. The whole length of the conduit lies in gravel, and on this account the section has side slopes of 1.5 to 1. The working level of water in the conduit or canal is very nearly that of the original surface of the ground for its whole length, and the embankment was raised for the whole length. On account of the nature of the material the channel is lined with concrete for the whole prism, which does not only prevent leakage and possible breaks, but materially reduces the loss in head due to friction.

4. This channel is cut through rock, sand, and clay. The formation through which it is excavated is rather peculiar, owing

to the great tilting of the rock, the rock surface appearing as a series of saw teeth, the intervening spaces being filled with clay, sand, and gravel; through the rock section the conduit is 72 ft. wide, and in the earth 40 ft. wide on the bottom and 80 ft. wide at the top; it is approximately 650 ft. in length.

Useful Information

SLOPE OF REPOSE AND WEIGHTS OF LOOSE EARTH (EARTH CHANNELS)

Kind of Earth.	Slope of Repose.	Angle of Repose.	Weight in lbs. per Cubic Foot.
River mud — — —	...	0° 0'	90
Ordinary soil — — —	1.33 : 1.0	36° 53'	90
Clean sand — — —	1.50 : 1.0	33° 42'	95
Dry clay — — —	1.33 : 1.0	36° 53'	100
Sand and clay — — —	1.33 : 1.0	36° 53'	100
Damp plastic clay — —	2.00 : 1.0	26° 34'	100

Water-Supply Pipes.—The approximate loss of head in a pipe, in feet, can be computed from the expression—

$$h = \frac{L(V^3 + 1.25V - 0.5)}{250d}$$

where

d = diameter of riveted pipe in inches.

V = velocity of flow in feet per second.

L = length of pipe in feet.

The thickness of riveted steel pipe necessary to withstand water-hammer can be expressed—

$$t = \frac{(p' + p)r}{15,000 e}$$

where t = thickness of shell in inches.

p' = intensity of internal pressure in lbs. per sq. in.

p = additional internal pressure allowed for water-hammer.

r = radius of pipe in inches.

e = efficiency of joints.

Tail-Water Velocity.—The velocity of tail-race water usually ranges between about 3.0 and 4.0 ft. Velocity of water in suction-pit ranges between about 1.5 and 2.0 ft.

Piers for pipe lines should be spaced 20 to 40 ft. apart, depending on the size and slope of pipe and the nature of hillside materials.

CHAPTER VII

PRESSURE PIPE LINES

IN attempting to describe certain aspects of the most economical electrical transmission line design (see Chapter XII.), the author points out the unnecessary step of seeking extreme refinement in the calculations when the problem is based on a *fixed* set of assumptions as to the power, time-period, diversity of peak loads, power factor, specific cost of energy, and other considerations, all of which are more or less variable. A pipe line or penstock design is based on almost the same general theory. In the one case, the typical problem presented is : (a) Given a transmission line of fixed length, given voltage and given power, it is required to determine the size of the conductor so that for a given or assumed power loss in the transmission system the annual cost of the transmission system will be a minimum. And in the other case, the typical problem is : (b) Given a pipe line of fixed length working under a given static head with a given discharge, it is required to determine the size of pipe so that for a given or assumed power loss in the pipe line the annual cost of the pipe will be a minimum. Of the two, a much greater margin can be allowed in the latter, due to water-hammer, etc.

In this text, *pressure pipe lines* (taken as one class of hydraulic conduit) are discussed principally from the viewpoint of the hydro-electric engineer. The pressure pipe line is a closed pipe line or tunnel conveying water under pressure from : (1) The head-water to the forebay, or the surge-tank ; and/or (2) the forebay or surge-tank to the turbine. From the viewpoint of comparison, it is of interest to note, for a few well-known hydro-electric developments, the wide variation in the matter of choice of (*d*) and (*t*) for different pipe installations (see following table).

In actual practice it fortunately happens that errors in design of even 10 per cent. are in most cases not very important, provided they always occur on the side of safety ; obviously, where possible, such errors should be avoided. As an example of the amount of difference in design, the following list of actual installations is given for comparison :—

EXAMPLES OF EXISTING RIVETED PIPE LINE INSTALLATIONS

(Values taken from Technical Descriptions in the Press)

Location.	No. of Pipe Lines.	Length in Feet.	Head in Feet.	Diameter in Inches. (<i>d</i>).		Thickness in Inches. (<i>t</i>).	
				Max.	Min.	Max.	Min.
San Joaquin —	1	2,362	1,411	40	34	.866	.55
Jordon River —	1	6,332	1,164	36	30	.551	.31
Arizona — —	1	2,526	1,017	36	32	.665	.51
Nevada — —	1	3,280	984	24	24	.511	.32
Rjukanfos —	10	2,296	972	79	49	.984	.35
Tax Pango —	2	1,509	557	63	51	.590	.39
Katsuragawa —	6	853	328	66	58	.433	.31
Lavras —	1	538	105	24	20	.236	...
Rio de Janeiro —	1	351	98	67	63	.394	...
Rio de Janeiro —	2	73	33	39	39	.314	...

The ordinary theory of the flow of water in pipes on which practical formula are based, assumes that the variation of velocity at different points of any cross-section may be neglected. If the motion is steady the velocity at each cross-section remains the same, and if the cross-section area A is constant the velocity V at all sections must be the same; hence, the motion can be taken as being uniform.

Let Fig. 17 represent a section of pipe of length dL between cross-sections at H and $H + dH$ ft. above any horizontal datum line xx , the pressure at the cross-sections being P and $P + dP$ lbs. per sq. in. Also, let Q be the volume of flow or discharge of the pipe per second, A the area of a normal cross-section, and w the perimeter of the pipe. That is, cubic feet per second Q will flow through the space (the weight of water = $(62.4)Q$ lbs. (but let $62.4 = g'$)), falling through a height $-dH$ ft. The work done by gravity is then $-g'QdH$; a positive quantity if dH is negative, and vice versa. The resultant pressure parallel to the axis of the pipe is $P - (P + dP) = -dP$ lbs. per sq. in. of the cross-section; the work of this pressure on the volume $Q = QdP$.

The only remaining force doing work is the friction against the surface of the pipe. The area of that surface is $w dL$. The work expended in overcoming the frictional resistance per second is

$$\eta g' w dL V^2 / 2g \text{ (wherein } \eta = \text{coefficient of friction),}$$

or, since $Q = AV$,

$$-\eta g'(w/A)Q(V^2/2g)dL$$

(the negative sign being taken because work is done against a resistance).

Adding all these portions of work, and equating the result to zero, we have

$$-g'QdH - QdP - \eta g'(w/A)Q(V^2/2g)dL = 0,$$

dividing by $g'Q$,

$$dH + dP/g' + \eta(w/A)(V^2/2g)dL = 0.$$

Integrating, we have

$$H + P/g' + \eta(w/A)(V^2/2g)L = \text{constant}.$$

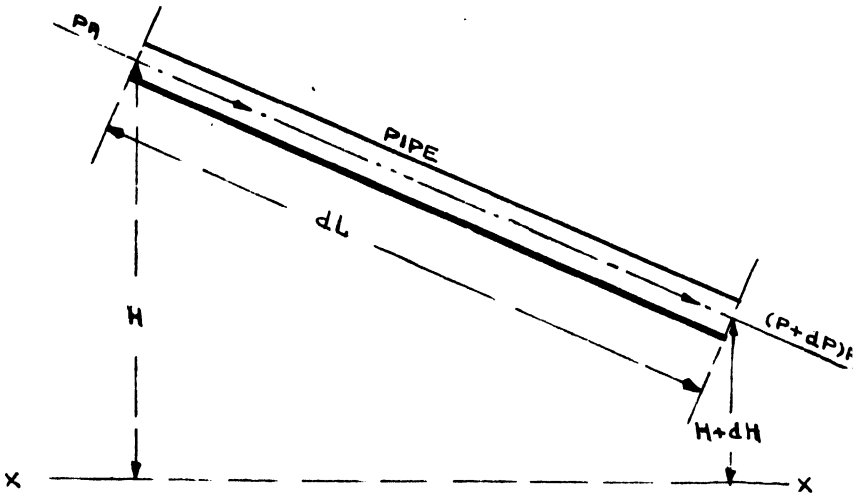


FIG. 17.

The quantity A/w is called the hydraulic mean radius of the pipe, thus for pipes of circular section and of diameter d , we have

$$A/w = 1/4\pi d^2/\pi d = 1/4d,$$

then

$$\eta V^2/2g' = (A/w)(h/L) = (A/w)S$$

or

$$h = (4L/d)(V^2/2g'),$$

which shows that the head lost (h) in friction is proportional to the head due to the velocity, and can be found by multiplying the head by the coefficient $4\eta L/d$ (see also p. 117).

The factor L of a pipe line affects the variation of head H directly and linearly, but the velocity V of the water affects the variation

directly as the square of the velocity (V^2). The influence of both size and length of a pressure pipe line upon the pressure and speed-regulation of a turbine is most pronounced, and the characteristics of a pressure pipe line should never be decided independently of the governing characteristics of the turbine. As an illustration, it is estimated that for a length of pipe of 600 ft. a maximum velocity of 20 ft. per sec. would require 5 secs. closing time to avoid pressure rises exceeding about 32 per cent. To reduce this pressure variation we would have to reduce the velocity of the water in the pipe line, but to do so would necessitate a larger diameter of pipe, resulting, perhaps, in prohibitive cost of the pipe line; a pressure regulator or relief valve could be used to prevent an abrupt change in velocity of the water. In the design of a pipe line it should be so dimensioned that (a) the velocity of water does not rise above the limit; and (b) the strength of the pipe itself must be sufficient. The velocity of water which should be allowed to flow will depend upon the total loss of pressure as well as upon the rise in pressure due to water-hammer and other surges which the pipe is able to resist in case of sudden closing of the gates or nozzle. The rise in pressure by sudden closing of the gates will increase rapidly with the velocity. If conditions are very favourable and the pipe line very short, a maximum velocity of $0.1 \sqrt{2gH}$ may be employed, but in cases of very high heads it is not possible to take full advantage of this rule. It is fairly safe for heads between 35 ft. and 100 ft., but if the pipe line is long, the velocity must be reduced accordingly in order to save friction losses. Near the distributing or "header" pipe the velocity may be increased from 5 to 22 ft. for higher heads and for high pressures and spiral casing turbines. This latter part of the pipe line (near the turbine end) is the most critical, because shocks are generally heaviest there.

Although, as explained later in the text, care should be taken in considering the *proper* pipe line, such as the size with respect to maximum power to be developed, the maximum pressure, and the suitability of the pipe joints, etc., there is the matter of *proper* location of the pipe line. When deciding upon a pressure pipe line it is necessary to :—

1. Make sure that the hillside is good.
2. See that the pipe line is as straight as possible, *i.e.*, free from bends, sharp drops, etc.
3. See that the pipe is given the best slope possible.
4. Have the pipe secured against any movement caused by the action of the water, *i.e.*, see that it is securely anchored and well backed, and that supports are properly spaced.

5. Take every precaution as to its being safely supported.

6. Ascertain that the downward pressure of the pipe line does not cause injurious pressure on the distributing pipe or to the powerhouse structure or turbines.

7. Make proper allowance for expansion, as great changes in temperature can cause leakage, loosen supports and anchorage, etc., and thus endanger the whole pipe line and the hillside.

8. See that all pipe supports and their foundations are good.

9. Give the pipe line a good clear right-of-way and every protection, *i.e.*, see that it is protected from the direct rays of the sun (reference to tropical countries) and from freezing (reference to cold countries), also from surrounding dangers, such as falling trees, rocks, etc.

10. Bear in mind that the better the pipe is laid, the less will be the need for pressure relief and other protection (internal and/or external).

In this text *riveted steel pipe* is the class considered. For all ordinary pipe-line design the working stress s in lbs. per sq. in. often allowed is 9,000, but where there is a danger of water-hammer, this working stress may be suited to the particular case. This value s (9,000 lbs. per sq. in.) and joint efficiency e of 100 per cent. forms the basis of Fig. 10; for other values, multiply H values in the diagram by $s'e/9,000$. Steel pipes are made of rolled plate, riveted together or lap welded. Lap joints are made single, double, and triple riveted, according to the static head. Butt strap joints are made double, triple, and quadruple. The lower sections of pipe lines working under high head may be of butt strap riveted pipe, or of seamless welded pipe; the latter pipe is used to avoid additional loss of head that would be caused by the riveted pipe, which latter has a greater number of rivets in the lower sections; the use of welded pipe has been retarded mainly because of uncertainty of the weld. Steel plates can be obtained in steps of $\frac{1}{16}$ in. thickness. In making a pipe-line design, $\frac{1}{16}$ in. is usually added to the calculated thickness. The minimum thickness of shell (tube) advisable is ordinarily between $\frac{1}{4}$ and $\frac{1}{2}$ in.; the actual thickness is, of course, proportional to the diameter of the pipe. Steel pipe has an expansion of about $\frac{3}{4}$ in. per 100 ft. at 100° F. For a straight steel rigidly held pipe line located above ground, expansion joints are usually required. Masonry pipe-line supports are often damaged by expansion and contraction of the pipe where adequate expansion joints are not employed; to avoid this trouble they are placed underground, or steel piers (set in concrete in masonry bases or in rock foundations) are employed, and an expansion joint is suitably fitted.

The general formulæ for determining the strength of riveted steel pipe is

$$t = \frac{Pd}{2T_s e}$$

wherein

d = diameter of pipe in inches.

P = pressure in lbs. per square inch.

k = factor of safety, say 5, based on the ultimate tensile strength.

e = efficiency of riveted joint ; 0.6 for single, 0.7 for double, 0.79 for triple riveted, etc.

T_s = tensile strength ; 50,000 for mild steel, 60,000 for wrought iron.

The expansion of steel pipe (increase in inches per 100 ft. of pipe) is approximately :—

TABLE XIV.—GIVING THE EXPANSION OF STEEL PIPE

Expansion.	Temperature.	
	(°C.)	(°F.)
0.38 in., at	10	50
0.76 „	38	100
0.92 „	52	125
1.15 „	66	150
1.34 „	80	175
1.57 „	94	200
1.78 „	107	225
1.99 „	121	250

Much importance is attached to the location, laying, and anchoring of a pipe line. The bottom end of the pipe line should be held securely to prevent forces due to temperature changes and other causes from throwing the turbine out of alignment. The gradient of a pipe line is rarely uniform. When a pipe line is laid for a considerable distance on a very slight gradient and then descends to the power house on a steep grade, its safety is jeopardised because the water flowing through the steep grade of pipe line will have a tendency to increase its speed much quicker than the water in that part on the slight gradient ; an air-valve or stand-pipe (preferably the latter) may be found necessary in such cases ; its location would be at the junction of the two gradients. Formulæ for determining the approximate maximum difference between the external and the internal pressures in lbs. per square inch which a steel pipe can withstand is

$$P' = k \left(\frac{t}{d} \right)^3 ;$$

wherein

d = diameter of pipe in inches.

t = thickness of shell or plate in inches ; and

k = approximately 50.2 millions.

All points of a pipe line should be well below the hydraulic gradient, otherwise the pipe may not stand a heavy overload without

danger of the water column parting on the summit. The pipe should be laid in the shortest distance to the power house and in such a manner as to avoid sharp angles, sudden changes in gradient, and abrupt changes in area. The value of the coefficient of roughness η of pipe is generally between 0.011 and 0.015; with corrosion and tuberculation, the value of η may rise to 0.020 or even more. And the head loss is often appreciably greater for an old pipe than for a new pipe.

In medium and high-head developments the final passage of the water before reaching the turbine is through penstocks or pressure pipes. The very low-head developments are usually completed without penstocks or pipes, as the turbines can be fed directly

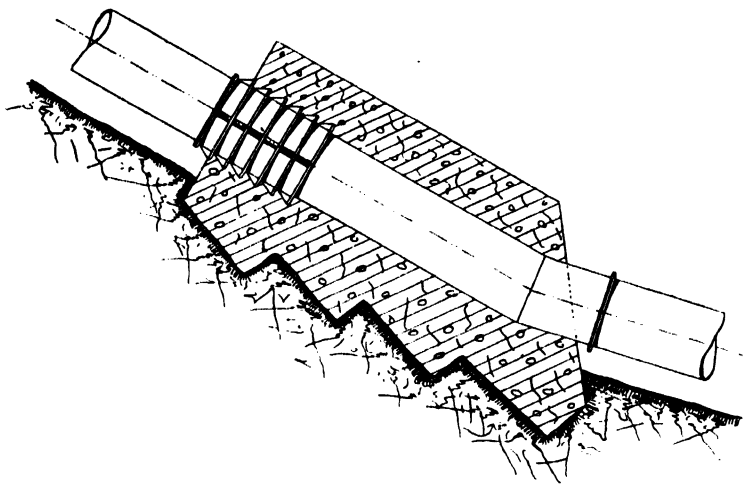


FIG. 17A.—Anchor-block of a pressure pipe line.

from an open flume or forebay. In the ordinary low-head development a reinforced concrete pipe or concrete-lined circular tunnel is sometimes employed, but more often the steel pipe is used. For nearly all heads, *steel*, in preference to all other material, is generally employed for pipe lines. Hence the reason why this class of pipe, *i.e.*, lap riveted and butt strap riveted pipe, is the only class covered by the text. Of course we may use other joints, such as the "muff," "bump," flange, etc., but never the lead joint; much use has been made of the angle and other types of flange joints.

The purpose of a pipe line is to conduct and convey energy in the most efficient manner from the forebay or from the reservoir, diversion basin, or from the head-race to the power house and turbine under certain specified conditions, and therefore it is important to locate, design, and construct the pipe line in the most reliable and economical manner. The importance is more intensified in a pipe line of great

length in which the capital outlay is comparatively large with respect to the total cost of the development. As in the case of a determination of the most economical transmission line, it is equally evident that while the method of calculation remains true, industrial and financial conditions change the value of energy and the cost of the installation, etc., and the constants *assumed* in the calculation to determine the economical condition may differ at one time from those assumed at another time. The principle of economy in the determination of the economical size of pipe line for a hydro-electric plant may be, and sometimes is, different for different plants, differently situated, and offering different characteristics. The economical design of an energy conduit may take several different forms, depending on requirements and the economic aspect. For instance:—

(a) It may be required to develop so as to yield a maximum amount of power irrespective of first cost, because of an unlimited power market. In such a case, maximum efficiency is required, and the economical size would be that giving the least total loss of head and water. On the other hand—

(b) Due to conditions offered, we may find it better to develop the scheme so that the annual cost of operation will be a minimum for a maximum output, and the annual net income a maximum. But—

(c) Where the working head and water supply are ample and the power market very limited (relatively so), then the lowest practicable minimum first cost per horse-power developed may be the best requirement. However—

(d) The usual requirement—where the amount of water available is limited—is to produce as much power as possible consistent with a fair return on the investment. Of course, when the water consumed has little or no value, it is allowable to employ the smallest diameter of conduit; this latter condition is normally not applicable to the text.

For low-head installations the pipe line is usually made to consist of a constant diameter and thickness throughout. With medium-head installations it is generally found more economical to make the pipe line consist of two or more sections, each section of the same diameter but different thicknesses of shell. With high-head installations it is more economical to make the pipe line consist of several sections, each section of different diameter and different thickness of shell, as shown in *Example II.* of this chapter. The diameters of steel pipes (of varying thickness of shell) for high-head installations vary nearly inversely as the seventh root of the head, *i.e.*, between 7 and 7.25. For low-head, the diameter will vary nearly inversely as the sixth root of the head, *i.e.*, between 6 and 6.25. For high-head installations it is generally found that much saving in metal and consequent cost can be made by tapering the diameter of the pipe in such a way that the total head lost in friction will

be the amount considered permissible, rather than by selecting a uniform or constant diameter which will give the same total friction head loss. If in each section of a pipe line the diameter of pipe decreases as the corresponding thicknesses increase with increasing head, the pipe line will more nearly approach the theoretically most economical pipe line.

In the design it is usually desirable to satisfy the commonly understood fundamental rules of economy by making a determination of the size of pipe so that the percentage return on the initial investment shall be a specified amount or a maximum, and so that the total net annual income shall be a *maximum*. Before designing a pipe line we require to know : (1) The general profile and topography of the site ; (2) probable drainage modifications due mainly to the location of forebay spillway ; (3) the character and conditions of hillside which must satisfy pipe anchorage and the most economical location of the forebay and the pipe line ; (4) the head under which the pipe line is to operate ; (5) the quantity of water available ; (6) the length of sections and of pipe line ; (7) best and most economical types of joints ; (8) character of loads ; (9) diversity of peak loads ; (10) load factor and other considerations. The principal quantities to be determined—after selecting the site—are : (a) the economical diameters for the different sections along the pipe line ; (b) selection of the best and most economical types of pipe joints ; (c) the thicknesses of the different pipe sections. The effect of joint efficiency on economy is extremely important. Of all the joints in use there are six common types of riveted joints which vary in efficiency from 57.0 per cent. up to 94 per cent. Pipe with welded longitudinal seams with either riveted or flanged circular joints has the advantage of minimum weight and low friction losses. Where reliable welding can be done, *the welded pipe is recommended for high-head installations*. In calculations where a formula is used for determination of the economical diameter, the required diameter of the pipe is unknown, and must first be assumed ; furthermore, according to the head, various types of joints must be given, and allowance should be made for weight which does not vary at a uniform rate with the diameter of pipe, etc. It is considered by the writer that a graphical presentation would be more practical than the formula itself. In the first place, to determine the economical size of pipe it is usually necessary to calculate a series of pipes and determine which is the most economical. By formulæ this is tedious, and results may be uncertain unless cross-checking of the calculations is done. By means of graphs or diagrams (such as those given herein) an error is visualised ; furthermore, it is quite a simple matter first to select a number of possible diameters for a given rate of flow (Q) with different assumed losses (h) and find *the* economical size from the

diagrams. For a high-head development the diameter would not be constant throughout the whole length, and therefore for a given loss at the start of the calculation it would be proportioned to different sections of the pipe line. Quite often the limitation of a minimum thickness of shell will indicate a pipe of large diameter at the top, thus allowing most of the total loss by friction to be at the lower end of the pipe line. Thus, by the use of Figs. 18 and 19 all these values can be set in profile, in tabular form or graphic form, and usually a more satisfactory solution can be obtained than by formulæ alone. As several different aspects of the same problem can be considered, it would seem better to lay out on paper one or several lines in profile (showing lengths, thicknesses, and heads), then find the losses and costs from the diagrams, and from this information determine the most economical size. The design may seek equal strength at all points on the pipe line in respect of either a new pipe or an *old* pipe, that is to say, after a lapse of a period of years corresponding to the normal life of the pipe. The losses and cost to be accounted for would generally be (a) head loss resulting from the pipe friction to the flow—the entry head and velocity head usually being omitted; (b) size and weight of pipe; (c) annual value of head loss; (d) cost of the installed pipe complete; and (e) annual charges on the total investment. The annual interest and depreciation on the initial cost of any section of the pipe line would be proportional to the amount of metal in the section, *i.e.*, proportional to the length of section; the diameter of pipe; the head on the point under consideration, and a *constant*. The location and fixing of the most economical sections would depend largely on the hillside conditions, the geology and topography, and therefore they affect the economic problem and must be considered.

In making the design, one of the first things to be done is to establish all relations between the cost and the several independent variables mentioned herein. The cost will have a minimum for a definite discharge and definite length by varying several of the independent variables. Should a pipe of relatively large diameter be chosen for the purpose of increasing the efficiency of the pipe line, the value of the extra amount of power obtained due to this higher efficiency may not merit the increased investment. On the other hand, should a relatively small size of pipe be chosen for the purpose of reducing the annual charges on the investment, the resulting loss due to the lower efficiency of the pipe may more than offset this reduction in annual cost. The energy loss accounted for in a pipe-line design is generally composed of one factor, namely, the energy loss due to friction. This loss increases with the length of pipe, is inversely proportional to the diameter of the pipe, and it

increases with the roughness of the interior surface of the pipe. In the economical design of a pipe-line system for a given flow, it is necessary to consider the number of pipes (separate pipe lines) and the amount of water which each pipe must be able to carry under specified conditions. For the larger schemes where large units are employed it is a common practice to have one pipe line for each turbine. The smaller installations usually cannot permit the expense of one pipe line per turbine, hence a distributing pipe or "header" is employed.*

In a pipe-line design there are several independent variables that enter into the economic problem. They are: (1) Number of pipe lines; (2) length of pipe lines; (3) total head and head losses; (4) factor of safety allowed; (5) size of pipe section or sections; (6) thickness of pipe section or sections; (7) available quantity of water; (8) velocity of flow at normal and peak loads; (9) nature and amount of load; (10) selling price of energy; (11) allowance for interest and depreciation of the investment, etc. The principal factors depend on the quantity of water, the static head, the cost per lb. of metal, and the total cost of the development as a whole. These variable factors define the basis relating to the actual cost of a pipe line for a water-power scheme. On the other hand, the size and weight of pipe, and the discharge and velocity of water, relate to the economical design of a pipe line (varying in respective ranges), and they may be taken as representative of the principal variables that govern the economical design.

The discharge Q of pipes of different internal diameter d but with the same hydraulic gradient S is usually taken to vary as the square root of the fifth power of the diameter $d^{5/2}$; that is, the number of pipes equal in capacity to any given pipe may be approximately determined from the equation $x = d^{5/2} \div d_1^{5/2}$, in which x = number of pipes of diameter d_1 equal in capacity to a pipe of diameter d , d_1 and d in inches. The most commonly accepted equation, upon which the following table is based, is

$$x = \left(\frac{d^5}{1 + \frac{3.6}{d}} \div \frac{d_1^5}{1 + \frac{3.6}{d_1}} \right) = \frac{d^3 \sqrt{d_1 + 3.6}}{d_1^3 \sqrt{d + 3.6}}$$

Example.—What is the relative discharging capacity of two smooth pipes, one 48 in. and the other 16 in. in diameter, laid on the same hydraulic gradient? The following table shows that a 48-in. pipe has 16.6 times the discharge capacity of a 16-in. pipe.

Also, a 42-in. pipe will carry as much as 11.9 times that of a

* See "Hydro-Electric Development in Bolivia," *Electrical Review*, 5th May 1922, p. 617.

16-in. pipe ; a 30-in. pipe will carry as much as 5.03 times, a 24-in. pipe will carry as much as 2.84 times, a 20-in. will carry as much as 1.78 times, and an 18-in. pipe will carry as much as 1.36 times that of a 16-in. pipe laid on the same hydraulic gradient.

This table and the equation may be found useful when choosing a pipe-line installation for a given total discharge. *For example :* It is seen, from equation, that a 48-in. pipe will discharge nearly twice as much as a 36-in. pipe. Hence, in practice, if two pipe lines are considered most desirable, then two 36-in. pipes may be required in the place of one 48-in. pipe. The cost of metal and other factors will, of course, decide whether it is best to install one or several pipe lines for a given water-power installation.

TABLE XV.—TABLE OF RELATIVE DISCHARGING CAPACITY OF PIPES

$$\text{Equation } \frac{d^3 \sqrt{d_1 + 3.6}}{d_1^3 \sqrt{d + 3.6}}$$

Diam.	2	3	4	5	6	7	8	9	10	12	14	16
48	...	...	...	...	...	...	...	75.0	56.8	35.2	23.5	16.6
42	...	...	...	...	...	...	73.0	53.4	40.5	25.1	16.8	11.9
36	...	...	...	...	...	70.4	49.3	36.1	27.3	16.9	11.3	8.01
30	...	...	...	...	67.6	44.2	31.0	22.7	17.2	10.7	7.14	5.03
24	...	...	...	62.1	38.2	25.0	17.5	12.8	9.70	6.01	4.02	2.84
20	...	...	...	38.9	23.9	15.6	10.9	8.02	6.07	3.76	2.52	1.78
18	...	...	54.1	29.6	18.2	11.9	8.35	6.11	4.63	2.87	1.92	1.36
17	...	...	46.6	25.6	15.7	10.3	7.20	5.27	3.99	2.47	1.66	1.17
16	...	...	39.9	21.8	13.4	8.78	6.15	4.51	3.41	2.12	1.42	1.0
15	...	75.3	34.1	18.7	11.5	7.52	5.27	3.85	2.92	1.81	1.21	...
14	...	62.2	28.2	15.4	9.48	6.21	4.35	3.18	2.41	1.50	1.0	...
13	...	50.7	23.0	12.6	7.75	5.07	3.56	2.60	1.98	1.22	...	...
12	...	41.6	18.8	10.3	6.34	4.15	2.91	2.13	1.61	1.0	...	...
11	...	33.1	15.0	8.22	5.05	3.31	2.32	1.70	1.28	...	...	...
10	...	25.8	11.7	6.40	3.93	2.57	1.80	1.32	1.0	...	...	...
9	60.8	19.5	8.85	4.85	2.98	1.93	1.37	1.0	...	...	...	...
8	44.5	14.3	6.48	3.54	2.18	1.43	1.0	...	...	...	...	...
7	31.2	10.0	4.54	2.49	1.53	1.0	...	...	...	...	...	...
6	20.4	6.56	2.97	1.63	1.0	...	...	...	...	...	...	...

The cost of a pipe line is dependent not only on the quality and the amount of metal and materials, but on their market conditions. Where conditions require that the factor of safety be very high, and in places where the market price for metal, etc., is very high, the cost of the completed pipe line may be excessive. Moreover, the annual cost of a pipe line is dependent not only on its original or its initial investment, but also on the coefficient representing interest and depreciation and other operating expenses. That is to say,

a low investment may very quickly become a large cost item when the coefficient of annual expense is large on account of high maintenance costs, etc. When the pipe line is long, the cost due to the head loss and that due to the annual expense of the complete installation may tend to increase almost proportionally. For very long pipe lines the former may be the predominating factor of the first cost, that is, the cost due to the energy lost in the pipe line, and the variation and character of load will have a direct influence on the *most* economical design and on the total value (power loss value *plus* interest value). When the demand for power is small at low load factor, the economical design would indicate a smaller pipe than in a case where the demand for power is large at high load factor.

To reduce the cost of a pipe line, a rather common fault is to make the pipe too thin where no great strength is wanted. The strength of riveted pipes should be such as to withstand from 55,000 to 80,000 lbs. per sq. in. pressure, depending on its location. The price of metal is not a constant for all diameters; it is usually less for large pipes of thin metal than for small pipes of thick metal.

Knowing the area of a pipe ($A = .7854 d^2$) (see Fig. 19) and the quantity of water flowing ($Q = AV$), the velocity V is obtained by dividing the quantity Q by the area A of the pipe, that is, $V = Q/A = Q/(\pi/4)d^2$. Thus, for a given quantity of water, the size of pipe can be determined (see Fig. 18) when the velocity or speed of the water is known. This velocity ordinarily varies for the different installations from a minimum of about 2 ft. per sec. to a maximum of about 7 ft. per sec., or even more, the *average* being about 4 ft. per sec. From Fig. 18 it will be noted that for a flow of 3 ft. per sec., the flow in cubic feet per minute about equals d^2 . Low velocity has a range of about 2 to 3.5 ft. per sec.; medium velocity about 3.5 to 5.5 ft. per sec.; and high velocity of water a range of about 5.5 to 12 ft. per sec. For low speed of the water it is necessary to have low-head, large diameter of pipe, great length of pipe line, many bends in the pipe line, variable loads, and low load factor. On the other hand, high speed of water necessitates high-head, small diameter of pipe, short pipe line, few or no bends, steady loads, and high load factor. Judging from these two extreme conditions, it is evident that conditions arise in practice whereby several of these variables overlap. While a high velocity entails a considerable head loss, a low velocity necessitates a larger pipe, and this increases the cost. The pipe slope and velocity are both fixed by the volume of water to be carried and by the diameter of the pipe, that is, $V = Q/.7854 d^2$.

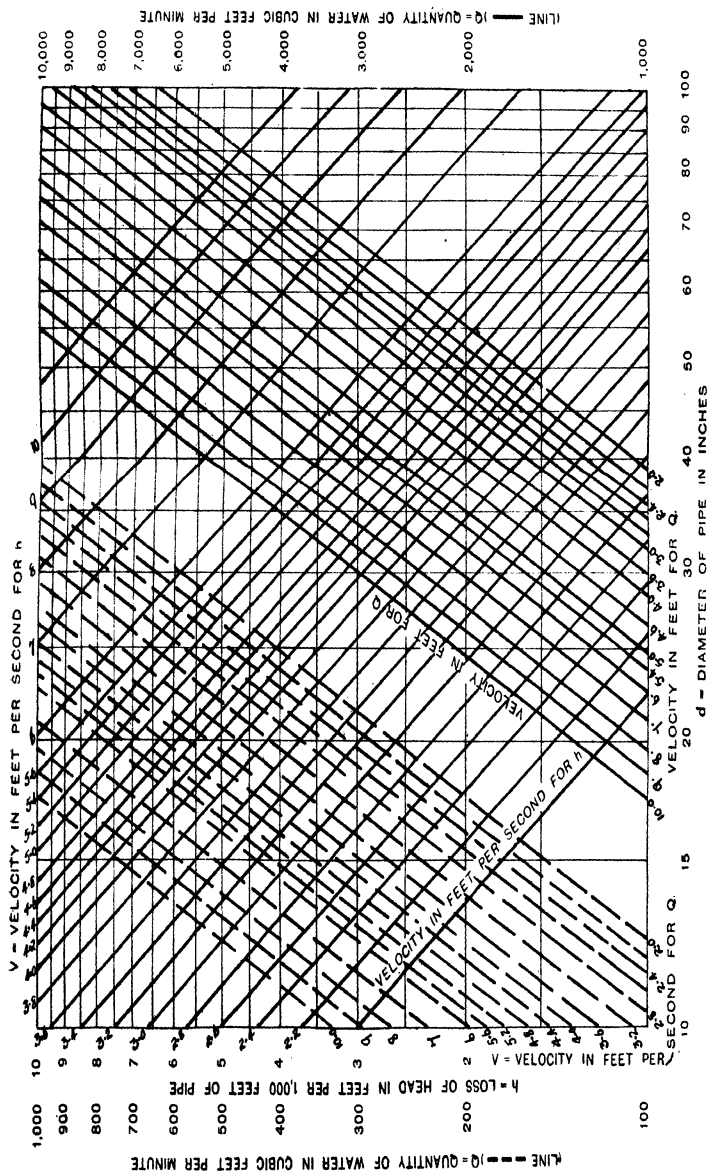


FIG. 18.—Loss of Head in Feet per 1,000 ft., and Discharge in Cubic Feet per Minute, for Riveted Steel Pipe.

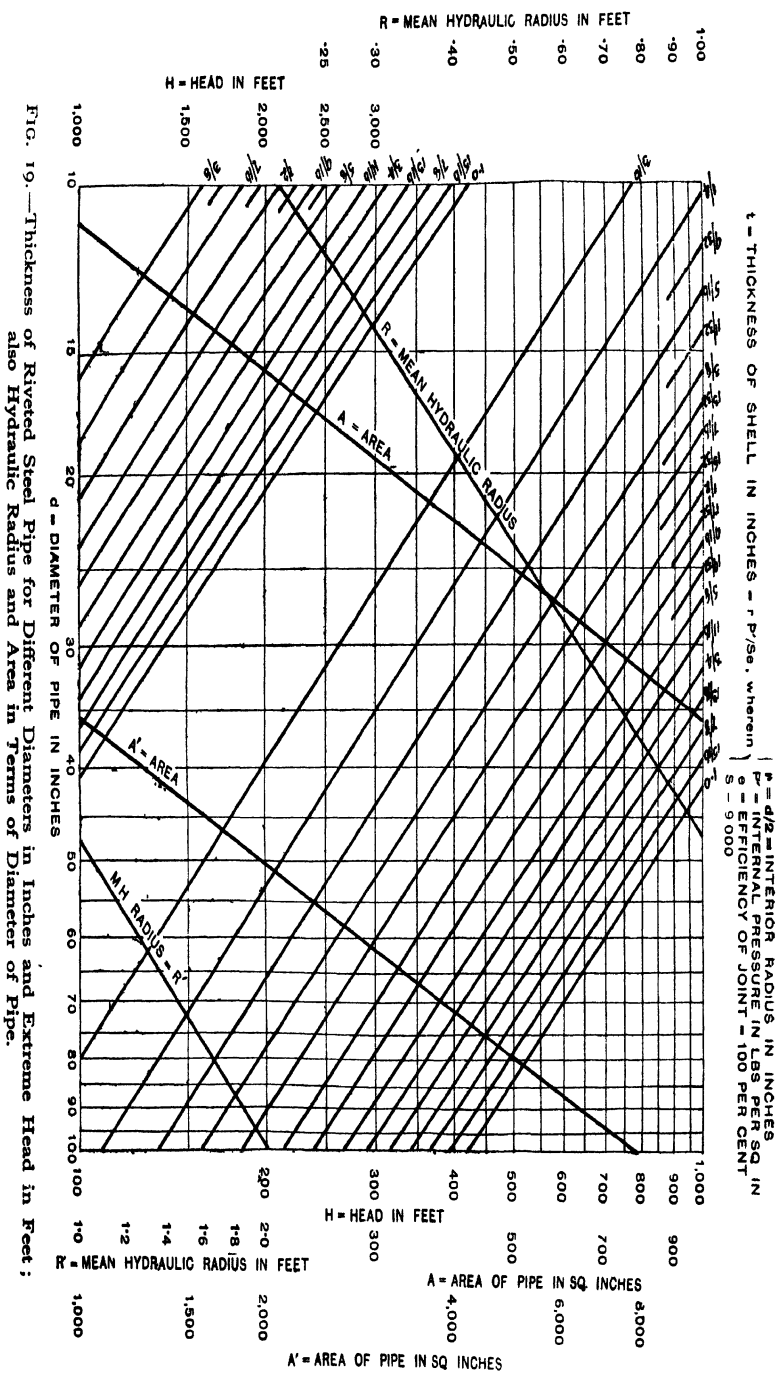


FIG. 19.—Thickness of Riveted Steel Pipe for Different Diameters in Inches and Extreme Head in Feet; also Hydraulic Radius and Area in Terms of Diameter of Pipe.

The approximate formulæ for the diameter of a *riveted* pipe is

$$d = \sqrt[5]{\frac{(Q)^2 l}{1,507 h}} \text{ ft.}$$

where

Q = discharge in cusecs.

l = length of pipe in feet.

h = friction loss in feet.

For a low-head installation a relatively low velocity should be used, because the loss of head will then form a much larger percentage of the gross or total head than for a high-head installation. In a high-head pipe line of great length it is more economical to use a relatively smaller diameter and greater velocity at the bottom of the pipe line where the pressure is higher and the pipe shell thicker. From Fig. 19 it will be seen that, with static heads less than 160 ft., the thickness of the metal required is less for all sizes than that necessary for durability and mechanical requirements alone. It is usual to allow a minimum of about $\frac{3}{16}$ in. thickness for pipe up to about 12 in. diameter, $\frac{1}{4}$ in. for pipe up to 60 in. diameter, about $\frac{3}{8}$ in. thickness for larger pipes up to about 100 in. diameter, and for pipes over 120 in. diameter a minimum of $\frac{1}{2}$ in. thickness of shell is usually employed. For a given head the thickness will depend on the type of joint. For instance, assuming an extreme static head of 250 ft. in each case, we have the following thicknesses of shell for different strengths of joints (see Fig. 19) :—

TABLE XVI.—SHOWING, FOR A GIVEN HEAD IN FEET, THE THICKNESS OF STEEL PIPE FOR DIFFERENT JOINTS AND SIZE OF STEEL PIPE (see also Table XVI A, p. 141).

Type of Joint.				(a) S.R.L.	(b) D.R.L.	(c) T.R.L.	(d) D.R.B.	(e) T.R.B.	(f) Q.R.B.
Diam. of pipe in in.	48	—		$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{9}{32}$
"	"	"	60	—	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{9}{32}$	$\frac{5}{16}$	$\frac{11}{32}$
"	"	"	72	—	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{11}{32}$	$\frac{3}{8}$	$\frac{13}{32}$
"	"	"	96	—	$\frac{5}{16}$	$\frac{7}{16}$	$\frac{15}{32}$	$\frac{1}{2}$	$\frac{17}{32}$
"	"	"	120	—	$\frac{13}{32}$	$\frac{17}{32}$	$\frac{15}{16}$	$\frac{5}{8}$	$\frac{11}{17}$

A simple rule for use in determining the thickness of riveted steel pipe for given pressures is

$$\frac{P(d/2)}{10,000},$$

where P = pressure in lbs. ; d = diameter of pipe.

Example.—A pressure of 108.5 lbs. or $108.5 \times 2.31 = 250$ ft.-head in a 48-in. pipe would require

$$\frac{108.5 \times 24}{10,000} = .25 \text{ in.} = \frac{1}{4} \text{ in.}$$

Wherein *a, b, c* are single, double, and triple *lap* joints respectively, and *d, e, f* are double, triple, and quadruple *butt* joints respectively. Obviously, the actual working head is 144 ft. for (*a*) and 235 ft. for (*f*). When high efficiency is desired, the lap or single *butt* strap joint should not be used. For such cases, double butt straps with three, or even four, rows of rivets on each side are often employed. The efficiency is thus raised from 57.7 per cent. for the single-riveted lap, and 74 per cent. for the double-riveted lap, and 75 per cent. for the triple-riveted lap, to 82 per cent. in the case of the double-riveted butt joint; 87.5 per cent. for the triple-riveted butt joint; and 93.7 per cent. efficiency for the quadruple-riveted butt joint. Thus the efficiency is increased from 57.7 to 93.7 per cent. respectively, or 36 per cent. increase. For certain equal pressure conditions, the quadruple-riveted butt is generally the cheapest type, while the single-riveted lap joint, from the viewpoint of economy for high and medium head pipe lines, is usually limited to the upper or the top section. In terms of relative safe working head, the advantages of the different types of joints can best be understood from the following table; thus, taking *equal* static head of 425 ft. :—

Type of Joint.	S.R.L.	T.R.L.	Q.R.B.
Diameter of pipe — — — —	36 in.	36 in.	36 in.
Thickness of shell — — — —	$\frac{5}{8}$ "	$\frac{1}{2}$ "	$\frac{3}{8}$ "
Cost per 1,000 ft. at 4d. per lb. <i>in place</i>	£5,000	£4,250	£3,500
£5,000 will purchase — — — —	1,000 ft.	1,176 ft.	1,430 ft.

Losses in riveted pipe lines produced by eddies are due to rivet heads in both circumferential and longitudinal joints, longitudinal butt strap ends, and abrupt changes in size of the ends of sections which require long, tapered reducers. Where sections change in diameter, long tapers are necessary to avoid losses of head due to impact and swirls. Head loss due to an enlargement is usually more serious than that due to an abrupt contraction. Usually welded joints are employed at the bottom section or sections of high-head pipe lines. In actual practice the *over-all* efficiency of pipe lines vary from about 2 to 10 per cent., the former value

for good design and the latter value for poor design and long pipe lines. The theoretical power from the water, conducted through and delivered from a pipe, is—

$$[(62.4)Q(H-h)] \div 550 = [(Q)(H-h)] \div 8.82;$$

wherein H is the gross head and h is the head lost due to friction, etc. Hence, *power loss* is—

$$P = (62.4)Qhe/550 = Qhe/8.82 = Qhe';$$

where

$$e' = e \times .1134 \text{ (h.p.)}$$

$$e' = e \times .10 \text{ (e.h.p.)}$$

$$e' = e \times .0846 \text{ (kW.)}$$

Example I.—The surveyed head is 400 ft. and the length of pipe line is 1,140 ft., the quantity of water available is 40 cusecs (2,400 cub. ft. per minute); the velocity of water in the pipe line will average 4 ft. per sec. for full-load conditions; a double-riveted butt strap pipe is to be employed. What will be the pipe diameter d , thickness of pipe shell t , the total head loss h , power loss P , and cost of pipe c , at 4d. per lb. of metal *in place*? From Fig. 18 we find at the intersection of $Q = 2,400$ and $V = 4$, that $d = 42$ in. diameter; we also find from $d = 42$ and $V = 4$, that $h = 1.7$ ft. head loss per 1,000 ft., which makes total head due to friction $1.7 \times 1.14 = 1.92$, or 2 ft. (round numbers).

From Fig. 19 we find, for a static head of 400 ft. and use of D.R.B. joint of 82 per cent. efficiency, the thickness for $H = 400 \times .82 = 328$ ft., and $d = 42$ in., that $t = \frac{5}{16}$ in., and, assuming a chance of water-hammer, allow $\frac{1}{16}$ more, making $t = \frac{3}{8}$ in. shell thickness. Fig. 19 is based upon 9,000 lbs. working stress S on the gross section and 100 per cent. joint efficiency. From Fig. 20 we find the weight per foot of pipe to be 227 lbs., and the cost $c = \text{£}3.15$ s. per ft. The total cost is $\text{£}4,310$.

Total power lost in the pipe line, assuming 80 per cent. efficiency, is $P = .09QH = .09 \times 40 \times 2 = 7.2$ h.p., hence the cost of pipe per horse-power lost is $4,310/7.2 = \text{£}60$. Also, assuming that power can be sold at $\text{£}4$ per h.p.-year, the annual cost of lost power would be $4 \times 7.2 = \text{£}29$; this, capitalised at 10 per cent., is $\text{£}290$. The gross horse-power is $0.1134QH = 0.1134 \times 40 \times 400 = 1,814$ h.p., but, at the turbine shaft, it is $.09QH = .09 \times 40 \times 400 = 1,440$ b.h.p.; the chosen size of generator is 1,000 kW. If the pipe line had been divided into several sections of, say, 42, 40, and 38 in. diameter respectively, and the pipe line had been in different thicknesses such as $\frac{5}{16}$, $\frac{3}{8}$ in., etc., a saving in metal could be made.

Example II.—Take now a case where the pipe line is tapered and the thickness of shell is allowed to increase in sections with

increasing head. In this example it is assumed that the turbine, when fully loaded, gives 3,720 b.h.p., but, from the actual amount of water required, the gross power involved is 4,650 h.p. for 2,400 cub. ft. per minute (40 cusecs). The surveyed head is 1,025 ft., and *one* pipe line is decided upon. In making a choice of the diameter, the mean speed of water may be taken at 4 ft. per sec. Hence the mean area of pipe for

$$A = Q/V = 40/4 = 10 \text{ sq. ft.} = 10 \times 144 = 1,440 \text{ sq. in.} = 42 \text{ in. diameter (Fig. 19)}$$

The mean hydraulic radius $R^1 = .0208d = .0208 \times 42 = .874$ ft. (Fig. 19). In this example we shall take the diameter as varying from 36 to 48 in., *i.e.*, about 3.0 in. per 1,000 ft. length of pipe.

The pipe line is to be divided into four sections (following the hillside topography), 1,200, 920, 600, and 700 ft. respectively in length, and with corresponding heads of 350, 500, 700, and 1,025 ft. respectively. Tabulating these values we have, from Fig. 18, the following results:—

Section Head.	Section Length.	Q.	<i>d</i> .	V.	<i>h</i> .	Total <i>h</i> .
0 to 350	1,200	40	48	3.3	.9	1.08
350 „ 500	920	40	42	4.2	1.8	1.656
500 „ 700	600	40	36	5.7	4.1	2.46
700 „ 1,025	700	40	36	5.7	4.1	2.87

Total *h* = 8 ft.

To this total head loss (*h* = 8 ft.) caused by friction in the 3,420 ft. of pipe, allow 2 ft. for additional friction due to rivet-heads, bends, and tapered reducers at the section ends, making *h* = 10 ft. From Fig. 19 we have:—

Section Head.	Type of Joint.	H.	<i>d</i> .	<i>t</i> .
0 to 350	D.R.B.	$350 \times .82 = 287$ ft.	48	$\frac{5}{16}$
350 „ 500	D.R.B.	$500 \times .82 = 410$ „	42	$\frac{7}{16}$
500 „ 700	Q.R.B.	$700 \times .937 = 656$ „	36	$\frac{9}{16}$
700 „ 1,025	Q.R.B.	$1,025 \times .937 = 960$ „	36	$\frac{13}{16}$

Fig. 20. Weir and Steel Pipe's complete including Weight of Rivets and Straps or Laps.

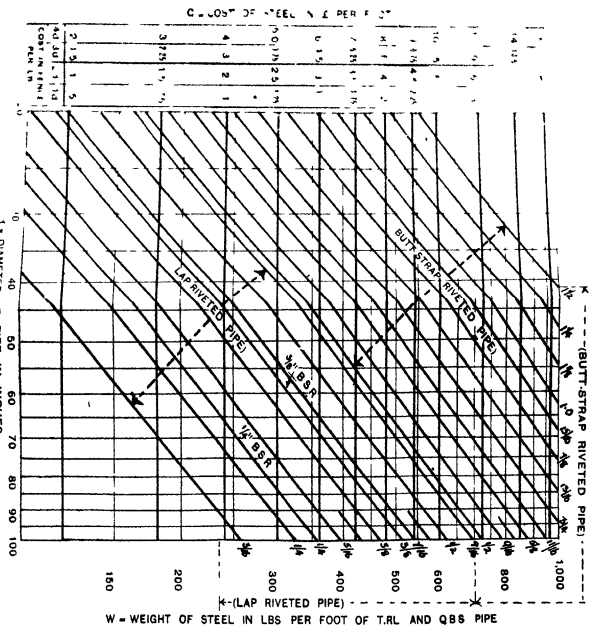
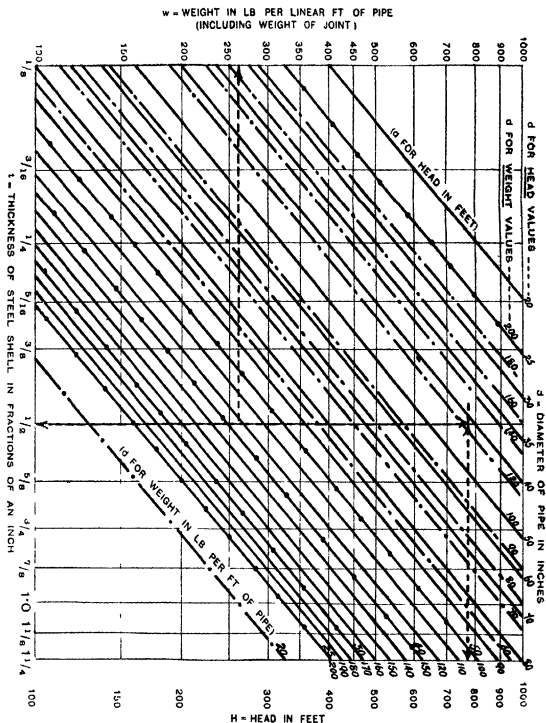


Fig. 21.—Thickness of Welded Pipe based on 14,000 lbs. Unit Stress on Gross Section and Joint Efficiency of 100 per cent.



$t = r/p/es$, where r = radius of pipe in inches, p = water pressure in lb. per sq. in. = $H \times 0.433$; e = efficiency of joint as compared with gross section (100 per cent.); s = safe stress on the pipe section in lb. per sq. in. (see Table 1, page 215).
 $t = r/p/es$ = $42 \times 0.433 / (100 \times 0.9) = 0.20$ in. (see Table 1, page 215).
 Take the following example: What thickness of $\frac{1}{2}$ in. wall stand, assuming 14,000 lbs. as the safe working unit stress on the section and flange-joint efficiency of 100 per cent. ? At the intersection of values 42 in. d and $\frac{1}{2}$ in. t , we read, on the right side of diagram, 775 ft. as the safe head in feet for any pipe of given diameter, for any value of r other than 100 per cent., and any value of e than 14,000 lbs. per sq. in.; multiply H as read by 775 ft. by 0.433, and divide by 100 per cent. of H , d , and t , and let e be 90 per cent. = 9, and $s = 12,500$, then safe head H is:

$$H(0.433/14,000) = 775[(12,500 \times 9) + 14,000] = 620 \text{ ft.}$$

From Fig. 21 we find total weight of metal in the welded-steel pipe line to be about 260 lbs. per ft. of pipe; see intersection of $d = 42$ in. and $t = \frac{1}{2}$ in. On the left side of diagram we find $w = 260$ lbs. per linear foot of pipe. Hence, for 1,000 ft. at 4.5 lb. per lb., the pipe cost = \$4,500. Fig. 21 for H is based on $t = r/p/es$; hence, for this example—

$$t = \frac{(42)(9)(620 \times 0.433)}{0.9 \times 12,500} = \frac{1}{2} \text{ in.}$$

and from Fig. 20 we obtain :—

Section Head.	<i>l.</i>	<i>d.</i>	<i>l.</i>	<i>w.</i>	Cost at 4d. per Lb.	Total Cost.
				Lbs.	Per Ft.	£
0 to 350	1,200	48	$\frac{5}{18}$	195	£3 5 0	3,900
350 „ 500	920	42	$\frac{7}{18}$	240	4 0 0	3,680
500 „ 700	600	36	$\frac{9}{18}$	318	5 6 0	3,180
700 „ 1,025	700	36	$\frac{13}{18}$	470	7 15 0	5,425

making a total cost of £16,185 for the pipe line.

The total power lost in the pipe line (assuming 80 per cent. efficiency) is $.09 \times 4 \times 10 = 36$ h.p.

Value of power lost, at £4 per h.p.-year, is $4 \times 36 = £144$.

Annual interest at 7 per cent. on investment equals £1,133.

Hence total annual value is the power loss value, *plus* interest value, of £144 + £1,133 = £1,277.

Different views of this same problem may be taken to determine *the* most economical size of pipe. We may estimate closer values of *d*, take two pipe lines in place of one, more pipe sections, other types of joints, different values for costs, for interest and for energy, etc. The above indicates a simple method, and one which need not necessarily be tabulated as shown here ; in fact, it is often better to draw a profile of the pipe line and mark in all the technical and physical data for different principles of economy, remembering that there are several, such as determination of the size so that the total net annual income shall be a maximum or that the percentage return on the first cost shall be a maximum, or that the percentage return on the first cost shall be a specified amount. Circumstances and general conditions should decide the best requirements for each pipe-line installation.

A rather recent development in the design of pipe lines has been the use of concrete for low-pressure lines where its low initial cost and durability give it a decided field of usefulness. In some places concrete pipes have been used as a pressure pipe line when a special form of reinforcement and treatment to exclude leakage has been adopted. Wood-stave pipe is also of value because of its durability, low cost, and low coefficient of friction, and it has often been used for the construction of the upper section of pipe lines. The welded-steel pipe lines signify one of the most important developments in recent years because of the progressive utilisation of very high-heads requiring close study of pipe-line design and

construction. Welded pipe lines are always used for high-pressure installations on account of their superior strength and absence of rivets which obstruct the flow of water. Welded pipes are heated by means of water gas and welded under high speed mechanically driven hammers which produce a weld of approximately 95 to 97 per cent. of the strength of the full plate. After welding, the pipes are annealed to remove all internal stresses. The foregoing process is only suitable for material up to about $1\frac{1}{4}$ in. thick, as above this thickness the heat would not penetrate sufficiently to produce a uniform welding heat. For larger plate thickness, the "wedge-welding" method is resorted to, the edges being brought together and a separate bar inserted, forming the weld. With this method pipes up to a thickness of $1\frac{3}{4}$ in. can be satisfactorily welded. The material used in welded pipe lines is best open-hearth steel, with a tensile strength of 56,000 lbs. per sq. in. and an elongation of 20 to 25 per cent. in 8 in. On account of pressure surges, plate thicknesses are calculated with a factor of safety of from 4 to 5, based on the strength of the weld equal to 100 per cent. Pipes are made in lengths of an average of 18 to 20 ft., using riveted joints for medium and fairly high pressures and flange or expansion joints for high pressures. In the riveted (so-called "bump") joint, the pipe ends are swelled so that the rivet heads do not obstruct the free area of the pipe. In high-head installations the *reinforced* welded pipe is sometimes used. This consists of a number of solid forged rings shrunk on to the outside of the pipe to add further strength to the pipe with a reduction of plate thickness. This innovation in pipe design permits, for high-heads, the use of a larger diameter pipe without exceeding the maximum limit of plate thickness to obtain a reliable weld, thus reducing the number of pipe lines for large installations and the initial cost of the installation. The value of this increase in diameter of pipe will be better understood on referring to the table of relative discharging capacity of pipes.

In connection with the installation of most pipe lines there are several auxiliaries to the line itself, such as valves, meters, etc., all of which have their special field of usefulness. The needle type of valves for high-head plants certainly call for special mention. In the *needle* type of valve the water passages are of liberal area, smooth in contour, involving no sudden changes in velocity, and free from pockets or projections. The velocity is either uniform or is gradually accelerated from the inlet to the outlet end, and the loss of head is practically negligible. The principal loss of head through hydraulic valves and other similar apparatus is due to whirls and eddies which result from sudden changes of area, obstructions, and other imperfections in design. All parts of the

needle valve are circular and hence free from distortion under high pressures. The simplicity of design and freedom from distortion make it possible to build the needle type of valve in sizes larger than could be attempted with any other type. They are in use and under construction up to 21 ft. diam. Some of the larger valves operate under heads up to 1,000 ft. The use of pressure pipe-line valves in water-power plants has become much more general than it formerly was, in fact, it is now almost universal, except in the case of very short pipe lines and low-heads. Gate valves and pivot or "butterfly" valves were extensively used in the past. Such valves cannot be operated successfully under conditions of flow in the pipe line, and a valve that can be used only when the pressures on both sides are equalised is so limited in its scope as to be entirely inadequate to meet the severe requirements which govern the operation of modern hydro-electric plants.

A reliable valve located close to the turbine, which can be closed under all possible conditions of flow in the pressure pipe line, is essential for the protection of the plant and the operation of the turbine and generator. The turbine is subject to dynamic stresses and shocks not common to the rest of the hydraulic system, and is much more liable to breakage. In such event experience has shown that the head gates at the entrance to the pressure pipe line can never be closed in time to prevent flooding the power house, and, even when closed, the entire contents of the pipe line must still pass through the plant. Furthermore, it is sometimes necessary, on account of mechanical trouble with the turbine, to shut off water quickly with the turbine gates open, and this cannot be done with head gates. At other times it may be necessary, on account of electrical trouble in the generator, to shut down quickly, and, if the turbine gates leak much, it may be impossible to stop the unit with the brakes, in which case it will run for a long time if there is no means of shutting off water other than head gates. Moreover, the proper inspection and maintenance of a turbine require a quick and easy means of shutting off the water to get into the casing. If head gates are used, the pipe line must be drained and refilled, which is not only injurious to the pipe line, but which is a long and tedious process. It is a serious matter to take a large unit off the load for so long a period, and it is done only when it can be put off no longer. This may result in serious damage to the machinery, and sometimes in a disastrous accident. On the other hand, if a valve is provided and the water-wheel can be shut down and inspected in a few minutes, the operator can go into the water-wheel frequently and on the slightest suspicion of trouble. Debris which collects in the turbine casing (cutting down the power and efficiency of the unit)

can be removed at frequent intervals. Another important point to remember is, when pipe-line valves are used, elaborate head gates may be dispensed with in many cases, as stop logs, or one portable gate serving a number of pipe lines, may be used. In this way a considerable part of the cost of the valve may be saved and a much better arrangement provided. It is a well-recognised fact that few head gates have ever been known to work properly when called upon in an emergency.

Pipe Line Installations

1. *40,000 h.p. Pipe Line.*—Two pipe lines, one for each machine, lead from a surge tank to the power house, which contains two 40,000 h.p. single runner, vertical-shaft turbines, each connected directly to a 35,000 kVA, 11,000 volt, three-phase generator. The diameter of each pipe line at the upper end is 129 in., tapering to 105 in. at the lower end. At each end butterfly valves are fitted, those at the surge tank end being the largest so far constructed. The pipe lines, each 1,357 ft. in length, are built in two sections. The upper end for a distance of 330 ft. is of the usual riveted steel construction, but for the lower 1,027 ft., pipe with forge-welded longitudinal seams are used. The pipe lines are laid on the surface of the mountain side, supported on concrete piers, and secured at seven points by concrete anchors.

2. *Pipe Line of 1,725 ft. Static Head.*—From the forebay the pipe line conveys the water down the hillside to the power house. The line was designed in two sections, two upper and eight lower pipe lines. The present installation (half the scheme) comprises one upper pipe line, a distributing pipe, and four lower pipe lines, the total length being about 13,000 ft., 8,200 ft. of upper and 4,800 ft. of lower pipe line.

The upper pipe line is built in three sections. The first section from the forebay is of riveted steel pipe 82.5 in. in internal diameter and $\frac{3}{4}$ in. in thickness; the second section is also of riveted steel tube 76 $\frac{3}{4}$ in. in internal diameter and $\frac{3}{8}$ to $\frac{1}{2}$ in. thick; and the third section (ending at the distributing pipe) is of lap-welded steel pipe 72 in. in internal diameter and $\frac{1}{8}$ to $\frac{1}{2}$ in. thick. The distributing pipe consists of a bend and a straight pipe 72 in. in internal diameter, $\frac{1}{2}$ in. thick, and about 46 ft. long, with four 42-in. branches and one spare 42-in. branch fitted with a blank flange and provided with a cover for a diameter of 72 in. to allow for connecting an additional future upper pipe line. The first section of the lower pipe line, starting from the distributing pipe, is of lap-welded steel tube 42 in. in internal diameter and $\frac{1}{2}$ to

$\frac{1}{8}$ in. thick ; and the second section (connecting up to the turbines) of lap-welded steel tube 38 in. in internal diameter and $\frac{7}{8}$ to $1\frac{1}{4}$ in. thick. Near the power house, exciter-turbine pipe-line connections consisting of three straight lap-welded pipes 15 in. in internal diameter and $\frac{9}{16}$ in. thick are provided between the 38-in. diameter pipes, with 10-in. diameter pipe connections to the exciter-turbine valves.

The riveted and lap-welded pipes are made from the open-hearth Siemens-Martin steel pipes having a tensile strength of from 28 to 32 tons per sq. in. and an elongation of 20 per cent. on standard test pieces for riveted pipes, and a tensile strength of 24 to 28 tons per sq. in. and an elongation of 26 per cent. for lap-welded pipes. The welded *joint* has a strength of 90 per cent. of that of the plate. The upper and lower pipe lines are designed for a factor of safety of 4.5 calculated on the minimum tensile strength and static heads.

The whole pipe line is laid above ground, proper provision being made for variations of temperature from 50° F. to 127° F. by expansion joints inserted between fixed joints in the upper pipe line and at each thrust block in the lower pipe line. Angle-iron rings are riveted to the pipes at the fixed joints and anchored securely to the masonry by means of iron bars, the thrust block being strengthened by the introduction of I beams in the concrete.

The waste-weir level of the forebay is 2,032 ft. and the power house floor-level 309 ft. above the mean sea level, giving a static head of 1,725 ft. to the centre of the turbine gate, with a pipe pressure of 743 lbs. per sq. in. With four turbines, each working at 11,000 b.h.p., the total loss of head is 66 ft. The pipe line is capable of supplying water simultaneously to four main turbines and two exciter turbines, with provision for connecting one additional main turbine to the upper pipe line.

3. 55,000 *h.p. per Pipe Line* (305 *ft.-head*).—From the forebay and screen-house the water is conveyed to the turbines in steel-plate pipe lines. The economical diameter of the pipe was determined to be 180 in., and in order to satisfy the field riveting the lower third of the pipe line was made 168 in. and the upper two-thirds 192 in. This decrease in diameter at the lower end permitted the use of plates $1\frac{1}{4}$ in. in thickness, and could be readily riveted in the field. The total weight of each pipe line is 840,000 lbs.

Each pipe-line ring is made up of two plates with longitudinal joints on the horizontal centre line, the plates varying from $\frac{1}{2}$ in. at the top section to $1\frac{1}{4}$ in. thickness at the lower section. These joints are all double-butt joints, varying from double riveted at the top to quadruple riveted at the lower end. The circumferential joints are made water-tight by electric welding. This type of circumferential

joint gives a very much better alignment to the inside of the pipe than can be obtained with the usual outside and inside course with lap joints. In designing the pipe line a stress of 12,000 lbs. per sq. in. was used, this figure being taken to provide for the exigencies of corrosion, fatigue, suddenly applied loads, and other intermediate or unknown contingencies. The internal pressure used for design purposes was taken to be the static head plus the pressure rise due to a complete closing of the turbine gates in $1\frac{1}{2}$ secs. The increase in pressure was taken as a maximum at the turbine gates, and varying uniformly to zero at the racks.

The pipe lines are covered throughout their entire length with a concrete envelope having a minimum thickness of 24 in., which protection will, it is believed, greatly increase the life of the steel pipes.

The entrance to each of the main pipe lines is a modified bell-mouth consisting of three openings 12 ft. 8 in. wide and 29 ft. high at the rack supports. These three openings gradually converge into one opening 16 ft. in diameter at the point of connection to the pipe line. In designing these water passages, particular care was taken to secure smooth stream-lines and consistent changes in velocity. The bell-mouth entrances are sealed by a concrete curtain wall which gives a depth of 28 ft. above the floor of the forebay. Immediately behind the curtain wall, steel-lined gate checks are provided to support structural steel gates. These provide a means of unwatering in case it is necessary to get at the lower sections of the racks, or for inspection of the pipe-line entrances. The pipe line for the service units follows the same alignment as the main pipe lines and has a diameter of 5 ft. As friction loss in this pipe was not such an important factor, lap joints and inside and outside courses are used.

Useful Information and Conversions

A column of water 1 in. square by 1 ft. high weighs 0.434 lb.

A column of water 1 in. square by 2 31 ft. high weighs 1 lb.

Water is at its greatest density at 39.2° F.

Sea water is 1.6 to 1.9 per cent. heavier than fresh water.

Inclination of Pipes, etc.—

1 in. in 1,000 ft. = very slow flow.

1 " 500 " = small supply pipes to reservoirs.

1 " 350 " = fall of ordinary rivers.

1 " 250 " = fall of rivers and rapid currents.

Mean hydraulic radius = mean area of cross-section (A) divided by wet perimeter = w .

= .25 diameter of pipe running full.

Sine of angle of slope = fall of water surface (H) in any distance (L) divided by that distance.

Area of a round pipe (inside) = diameter² × .7854.

Area in square feet = diameter in inches squared × .0054542 = A.

Diameter in inches = 13.5405 $\sqrt{\text{area in square feet.}}$

Diameter of pipe = 1.12838 $\sqrt{\text{area.}}$

Electrical horse-power = (cusecs × head in feet)/(8.82/.746).

= QH/10.5.

Kilowatts = (cusecs × head in feet)/(10.5/.746).

= QH/14.0.

1.0 cusec (ft.³/sec.) = 62.4 lbs. per second.

= 3,744 lbs. per minute.

= 373.3 gals. per minute.

= 3,600 cub. ft. per hour.

TABLE XVIA.—PERCENTAGE EFFICIENCY OF RIVETED JOINTS

Thickness (t) in Inches.	S.R.	D.R.	T.R.	D'.R'.	T'.R'.	Q.R.
$\frac{1}{4}$	57	72	80	81.6	88.0	94.6
$\frac{5}{16}$	54	70	77	80.8	87.6	94.3
$\frac{3}{8}$	53	69	76	80.3	87.5	94.2
$\frac{7}{16}$	51	67	75	80.2	86.3	94.0
$\frac{1}{2}$	50	66	74	80.1	86.1	93.7
$\frac{9}{16}$	48	65	73	80.0	85.3	93.5
$\frac{5}{8}$	...	...	...	...	85.7	93.0
$\frac{11}{16}$	...	...	...	...	84.2	92.5
$\frac{3}{4}$	...	...	...	...	84.0	91.0
$\frac{13}{16}$	...	...	...	...	82.8	90.5
$\frac{7}{8}$	...	...	...	...	82.5	89.0
$\frac{15}{16}$	...	...	...	...	81.7	87.8
1	...	...	...	...	81.0	87.2
$1\frac{1}{16}$	...	...	...	...	...	86.5
$1\frac{1}{8}$	...	...	...	...	...	85.5
$1\frac{1}{4}$	...	...	...	...	...	84.0

wherein S.R. = single riveted (lap).

D'.R'. = double riveted (strap).

D.R. = double riveted (lap).

T'.R'. = triple riveted (strap).

T.R. = triple riveted (lap).

Q.R. = quadruple riveted (strap).

Riveted Pipe.—For all ordinary cases it is better to take a factor of safety of 2.0 based on the elastic limit, than to take a factor of safety of 4.0 based on the ultimate strength.

IMPORTANT RIVETED PIPE INSTALLATIONS

Company.	Static Head in Feet.	Number of Pipes.	Length in Feet.	Size in Inches	Type of Turbine.	H.P. Capacity and Number of Units.
Pacific G. & E. Co. —	1,531	2	6,080	30	Impulse	2 of 7,500
" " " —	1,375	1	6,214	72-52	"	2 " 10,000
" " " —	577	1	2,565	42-36	Francis (single)	1 " 9,500
N. Californian Power Co.	487	2	3,600	72-60	"	3 " 7,000
Pacific Coast Power Co.	440	2	2,000	96-78	Francis (double)	2 " 20,000

CHAPTER VIII

HYDRAULIC LOSSES

It is now recognised that conclusions based on comparisons of water losses over a catchment area are, in general, as reliable, and sometimes more reliable, than either rainfall or run-off. If a study of the run-off and water losses of a given catchment indicates that these losses are abnormal, then a geologic study of the catchment should be made to find the leakage. Water leakage can be expected to occur on areas covered by deep beds of sand, gravel, and porous glacial deposits. Leakage also occurs through broken, seamy, and fissured rocks. The less impervious the soil and rock, the deeper the soil cover, and the gentler the slopes the greater will be the water leakage over a given catchment. In the case of a stream or an earth-lined conduit-section, where the soil is underlined with clay or impervious hardpan, there rarely exists a serious loss ; but where there is an underlying stratum of porous sand, gravel, or glacial deposit, losses of water are sure to occur and perhaps become a danger.

Losses by seepage and capillarity are of value in that they bear a relation to run-off. For such materials as peat and muck a high normal water content is usually in evidence, due partly to their humus composition and partly to the large relative pore space, accompanied by a high degree of capillarity. Part of the water held by saturated muck is released readily by drainage. The capacity of the material of the water-bearing stratum to transmit water depends mainly on the effective size of grain and on the porosity of the water-bearing material. The flow of water through almost any material varies approximately as the square of the effective size of grain ; the flow is greatly increased with an increase in porosity. The percentage of porosity varies in different materials and even in different parts of the same material, according to the relative size and arrangement of the particles (see p. 30).

Loss by leakage through the bottom and sides of reservoirs may have a great influence on the storage capacity—much will depend on the ground water level in the vicinity. Obviously, storage of water may be in a surface reservoir or underground. Usually, the proper use of a reservoir demands that it be operated in such a manner as to keep in storage the largest quantity of water

consistent with the necessary draft for use. Drafts should preferably be gradual and more or less regular.

Evaporation loss from the water surface of reservoirs and rivers is sometimes very great. The air over a catchment covered with forest, shrub, or green crops—largely protected from the direct rays of the sun—is much cooler than it is over country not so covered, hence it is better able to condense the heated and saturated air. Areas so covered are also effective in retarding run-off. In the study of run-off from any catchment it is desirable to look carefully into the relative proportion of water surface; this will usually involve the study of loss of water by evaporation. The water production from a known quantity of rain in a catchment embracing lakes or reservoirs of considerable extent, parts of which dry up during the dry period, may involve special study. For a given capacity, a wide and shallow reservoir will have a greater evaporation loss than one deep and narrow. Evaporation from water surface is dependent upon the same factors which control evaporation from soil surface, but in the latter it is also dependent on the degree of saturation of the soil surface. The amount of evaporation from water surfaces is mainly controlled by humidity, temperature, and wind velocity. One of the principal factors affecting evaporation loss is temperature. Temperature is also an important factor in relation to floods as affecting the rate of melting of accumulated snow.

With regard to estimating the probable run-off, it is thought possible that more satisfactory results may be obtained from approximate rating curves and gauge heights for limited periods than for studies of rainfall alone; strictly speaking, gauge heights are an index to area only. This question requires further investigation; much will depend upon conditions. It is also thought possible that, as periods of deficient rainfall have a retarding effect on the natural growth of vegetation, an examination of the annual rings of trees of sufficient age should give a fairly safe indication as to the occurrence of dry or drought periods (the periods framing the basis of water-power development). Based on rather incomplete investigations, it would appear to the writer that an examination of the occurrence of groups of narrow rings (indicating the annual growth, and, most likely, the very dry period) taken from trees differently located in the catchment would give additional data * when taken in conjunction with hydrometric work, and (or with) studies of rainfall. As regards stream flow, this is obtained in two

* To locate a "famine year" where the growth is almost nothing, it would probably be necessary to use a microscope to determine a ring of growth. During the year 1921 the writer used this method in British East Africa as a check on the then available hydrometric and rainfall records of certain catchment areas.

general ways only—*i.e.*, by computing run-off from general physical data, and by actual measurement of the flow. The measurement of rainfall and run-off is usually at best only approximate, for the reason that it is first necessary to obtain much reliable information, such as, for instance: (1) Rainfall: its intensity and distribution throughout the year; (2) total losses from evaporation, leakage, transpiration, and deep seepage, etc.; (3) temperature, humidity, wind velocity, and altitude; (4) geological, topographical, surface and subsurface conditions, etc.; (5) storage conditions, both surface and underground, and often other data. Thus, computed run-off from physical data is likely to be uncertain unless taken together with long-period, continuous records of the stream flow. Modern practice leans more and more to hydrometric work and to the plotting of data from stream flow records in the form of a continuous hydrograph, which can then be compared with other records in the same or in a similar catchment area.

Some of the more important hydraulic losses in the developed waterways of a water-power system are: (a) Loss due to high value of the coefficient of roughness; (b) excessive seepage in earth unlined sections; (c) loss due to grade; (d) losses due to obstructions, enlargements, contractions, and to changes in direction of flow; (e) losses due to leakage or waste; (f) general losses due to evaporation and seepage; (g) general losses such as those occurring in penstocks and turbines; (h) losses due to change in the head-water and (or) tail-water levels and to back-water conditions; and (i) those losses due to faulty or poor design, to bad construction, and to careless or improper maintenance of the waterways, plant, and equipment.

General formulæ for the flow of water does not account for these different factors; in fact, general formulæ accounts only for the head necessary to overcome friction. Actually, in addition to this head, account should be taken of the loss of head in creating the velocity of flow and in overcoming the resistance due to changes in direction of flow, obstructions due to bends, valves, and to enlargements and contractions, etc. Loss of head is expressed

in terms of $\frac{V^2}{64.4}$ multiplied by a coefficient. The developed water-

ways intake-loss of head is usually due to eddies, to friction, and to the design and shape of the entrance. Loss of head in the conduit itself is due to its length, its roughness, its design and shape, and to its construction and maintenance. The loss will increase with V^2 . Loss of head at the entrance to the pipe line (penstock) is due to interior surface friction, to its design and shape, and to the racks, screens, valves, gates, and other obstructions to the flow of water.

Neglect to clean the racks, etc., is usually a very great cause of loss. The loss of head in the pipe line itself is directly proportional to its length, inversely proportional to its diameter, and it increases with the interior surface roughness; it also increases with V^2 . The loss of head in the turbine depends on the type (impulse or reaction) employed; in the impulse type there is a loss caused by setting the turbine clear of the tail-water, thus failing to make full use of the free fall; in the reaction type, the loss of head is due to the length, the cross-sectional area, and to the shape of the "draft" or *suction* tube. Practically all these losses are capable of improvement with first-class design, good construction, and with proper care and maintenance. In fact, all the way from precipitation down to the stream itself, through the developed waterways, thence to the turbine, the wheel-race or the tail-race water level, a number of losses in one form or another are usually in evidence.

Apart from such controllable losses as leakage, seepage, obstructions, bad designs, and the like, losses due to stream conditions and slope (causing too quick a run-off) and due to grade of the developed waterways themselves, often form an important part of the total hydraulic losses. We may also find where water for regulating purposes is located—for instance, between a water conduit (head-race) and the power house—that, due to grade and/or *time-lag* of water, loss in horse-power-hour capacity of pondage may result; for pondage or storage of this character, time-lag of water may be of great importance, especially in the case of storage. Where storage is not used, loss due to grade of the head-race conduit itself, as affecting horse-power-hour capacity loss, may also constitute an important hydraulic power loss, the amount of loss depending on the length, type, and shape of the conduit, and on the character and condition of the conduit bottom and sides. Almost along the same reasoning as the latter (gravity conduit) there is a loss in the pressure pipe leading from the forebay or surge tank to the power house; this loss usually increases with the age of the pipe or conduit.

In the assumption or determination of the value of coefficient of roughness η it is sometimes found that far too low a value has been taken. Also, a not uncommon mistake is to take the same value of coefficient of roughness for a channel of identically the same design as regards grade, cross-section, and materials, neglecting the fact that the proper coefficient value should be different—that is, higher or lower, depending on the relative condition, etc., of the waterway. Thus, a channel with smoother and with more regular side-slopes and bottom, and one with less number of bends and with more uniformity of cross-section, should always be given a lesser value of η . The value of η is not by any means a stable factor.

There is, moreover, the stage or gauge height of the stream or (and) the depth of water in the conduit ; with particular reference to the former, as the latter is generally kept constant. That is, we may determine by measurements the value of η at normal stage of the stream, but, for other conditions of flow, a question may likely arise as to the use of this same coefficient value. For instance, in the case of a much lower flow than normal, there would most likely occur a greater loss due to eddies, ripples, and the like, and a portion of the measured cross-section at normal stage (normally used in computing the value of η) might result in quite a different value and appear much greater than that for the cross-section at normal or for a higher stage of the stream. In every case where practically the whole drop and cross-section are *effective* in producing velocity and conveying the water, the value given for η should be somewhat less than for other conditions of the channel and/or stage of the stream. With particular reference to rivers and streams, it is evident that the value of η should include: (1) Useless slope or fall or drop ; (2) cross-section ; and (3) a roughness factor due to the roughness of the channel sides and bottom. As the slope of the stream increases it becomes increasingly important that the coefficient of roughness η be correctly determined. If the slope is slight, a considerable increase in the coefficient of roughness may not produce any material change in the slope. For a constant discharge the slope varies practically as the square of η . For a fixed slope and fixed value of η , an increase in the hydraulic mean radius will result in a considerable increase in the velocity. For a developed waterway, the question usually arises as to a determination of the best or most economic condition for an increased first cost due to increasing the hydraulic radius and to the increased maintenance cost due to increasing the velocity.

Considering the developed waterways only, some of the suggestions toward a reduction in water and hydraulic power losses which the writer has in mind are : (1) More careful estimating of the stream flow ; (2) better knowledge of the net and gross static head ; (3) more efficient flashboard work ; (4) reduction of leakage and waste to a minimum ; (5) better use of surplus water, such as overflow ; (6) lining of waterways with concrete or the like, to decrease seepage and friction ; (7) designing of gravity conduits for best hydraulic radius—that is to say, with a very small wetted perimeter compared with the cross-section of the waterway ; (8) better maintenance of a higher forebay level ; (9) better water economy throughout the entire waterways ; (10) proper cleaning of the trash racks and the screens ; (11) most economical operation of waterwheels and turbines ; (12) operation of turbines and wheels at proper speeds

for maximum power and efficiency ; (13) proper wheel and turbine setting (with special reference to the reaction type) ; (14) elimination of all obstructions in the waterways, turbines, and draft-tubes and the tail-race ; (15) re-design of old systems and the installation of new units and systems to ensure that their designs are adopted to facilitate the many advantages and economical benefits now made possible by the greater experience and improvements in design, construction, and in the operation of water-power plants ; (16) the more economical loading of the power units ; (17) more concentrated effect to ensure higher efficiency not only of the units in individual power stations, but of the system or tied-in systems of which both the units and stations form an integral part ; and, finally, (18) to make a closer study into such questions as load factors of units and of the systems as a whole, the peak load demands, diversity, and similar considerations. These various items give only an indication or outline of the many important questions which tend to produce higher efficiencies. One cannot lose sight of the fact that, in arriving at a solution, usually it is not the high maximum efficiency but the high *over-all* efficiency that is most important and the principal object to be kept in view.

One of the characteristics of hydro-electric work is that it is almost futile to lay down fixed laws. The subject becomes one of where the conditions surrounding each particular case must be governed by its own law—whether it be the catchment, the stream, the developed waterways and plants, or the power load. The harnessing of, for instance, a large stream might involve complete regulation in the interests of flood control and of water supply, the stabilising of channels, improved navigation, better sewer outlets, more advantageous conditions for bridge piers, and better heads during flood conditions for low-head plants on the lower reaches of the stream. With reference to this latter question, the high and low water levels to be expected in the tail-race are extremely important ; it is also well to have in mind that the best and most efficient draft-tube design is dependent upon the proper elevation of the turbine above the tail-race water level.

An important problem of the future will be one of how efficiently water can be utilised rather than one of producing a reliable source of power, the latter having been accomplished. Higher efficiency will come from the study and training of run-off, and from the design, building, and maintenance of the development as a whole, commencing with the catchment itself, that is, by conservation and economical utilisation of the seasonal flow to the very best advantage and by getting as much of the total precipitation, in the form of run-off water and ground waters, to the turbines with the minimum of loss

of *water* and *head*. In other words, to obtain the greatest number of horse-power hours out of a given quantity of water or the greatest number of horse-power hours from the highest possible flow ratio value of run-off to total precipitation.

The principal losses in the head-race would usually occur in the unlined earth section of conduit. Losses in this type of conduit are of three kinds: those due to leakage, due to seepage, and due to evaporation. The loss from leakage is usually due to bad or improper design and/or construction. Evaporation loss takes place from both the exposed water surface and the moist soil on the channel banks adjacent to the water. Loss from seepage depends on the porosity of the material; it involves the depth of water, inclination of side slopes, and the ratio of depth to bottom width, that is, seepage loss increases with the depth of water and with the wetted perimeter. Usually evaporation loss in conduits is relatively very small compared with the loss from seepage *plus* leakage, and it is usually neglected. An increase in temperature of water increases the evaporation from it. The proper relation between cross-section and grade of a conduit requires the exercise of careful judgment. Other things being equal, the correct relation is that in which the velocity will neither be too great nor too small, and yet the amount of material to be removed will be reduced to a minimum.

The size of the forebay or regulating basin is generally governed by the topography of the locality at the head of the penstock, and as large a pond or reservoir as it is possible to build with reasonable expense should be made. If the forebay is not large enough, expense and trouble might arise, such as excessive wear of the buckets and blades of turbines; in other words, a forebay should allow for suspended matter such as sand, grit, etc., to settle, and it should also take up fluctuations in load. A forebay should therefore be large enough to equalise fluctuations in load, and the bottom should slope toward a sand trap so that, as the depth increases, the velocity of water is reduced, thereby facilitating the settling of and providing a suitable place for suspended matter. The function of a forebay or regulating basin is therefore twofold: (1) to maintain the head sensibly constant, and (2) to free the water from floating or suspended matter which would otherwise reach the turbine. A water-power plant with poor pondage, low-head and moderately long penstock, is practically limited by operating continuously only on the base load; whereas a water-power plant with good pondage, high-head, and relatively short penstock may be much over-installed and can be used on the peak load.

In computing water-power plant efficiency, the total or gross

head acting on the plant should always be taken; that is, the difference in elevation between the equivalent still water surface before the water has passed through the racks to the equivalent still water surface in the tail-race after discharge from the draft-tube. In computing the efficiency of the penstock, the losses in the turbines should not be charged against the penstock, nor should the head necessary to set up the velocity required to discharge the water from the end of the draft-tube be charged against the penstock or the turbines. In computing the efficiency of impulse turbine installations, it is charged with the net head with reference to the elevation of the centre of the nozzle taken as datum.

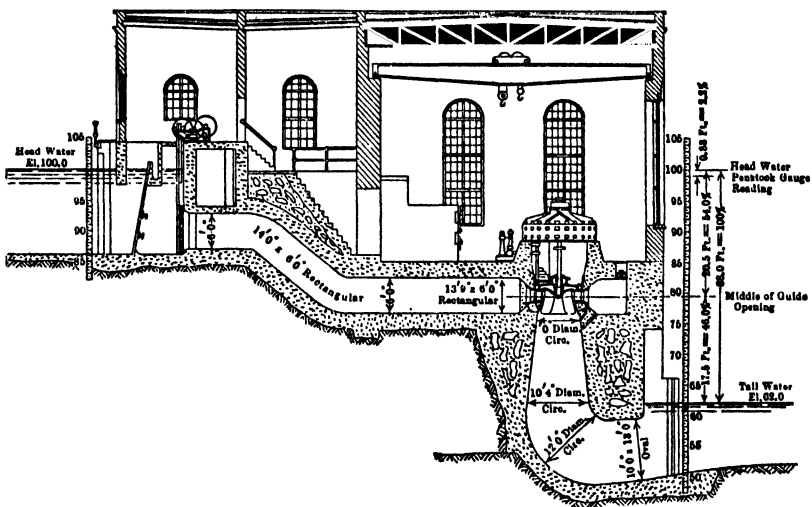


FIG. 22.—Showing low-head development of good design.

With reference to the tail-race and back-water conditions, in making a determination of back-water conditions, there is the uncertainty of finding the proper value of η . If η cannot be experimentally determined, it must then be assumed, and this is often where the trouble lies. The loss of effective head due to high tail-water may decrease the output of a power development more than the loss of head due to low head-water. A tail-race should preferably discharge parallel with the water's flow. Long tail-races are, as a rule, to be avoided for several reasons. The velocity of water in the tail-race should have sufficient speed to prevent it from "backing up." The tail-water velocity for medium sized low-head developments usually lies between 1.5 and 2.5 ft. per sec., but during flood conditions these velocities may increase as much as 80 per cent. or even greater. For higher heads up to about 800 ft., the discharge

velocity is usually limited to about 7 ft. per sec., on account of the great disturbance in the tail-race.

For the water-power development itself, commencing at the dam or head-work, or intake to the developed waterways, it is highly desirable that operating records be kept, giving conditions in all the component parts, sections, etc., making up the complete development. There should, for instance, be records of :—

- (a) Gate openings for various outputs and heads.
- (b) Gate openings for various quantities of water.
- (c) Records of drop in elevations for various quantities of water flowing through : (1) the intake racks ; or/and (2) the conduit ; or/and (3) the intake to pipe line (pointing out where loss of head is made up, *i.e.*, velocity head and friction head).
- (d) Records of quantity measurements at (1), (2), and (3), also records of quantity in the tail-race.

The information given by such records serves a very useful purpose, and may often be a means of solving what might appear to be doubtful operation problems. Apart from these advantages, there seems no reason why equally important metering and log recording should not be taken of the power development outside the generating station just as is done for the steam, hydraulic, and electrical plant, etc., inside the generating station. Electrical and other instruments are used for indicating and recording normal conditions and operation of a power plant. However, for the hydraulic end of the development, instruments will serve two very important functions, namely, the indication or recording of normal operating conditions as well as conditions differing very much from normal ; that is to say, the actual condition (normal and abnormal) of any section or component part of the hydraulic system of waterways can be indicated or recorded. The value of this information is not one of ordinary necessity for present or momentary requirements, as is the usual case with electrical plant. Past hydraulic records are always valuable. It is often necessary to compare past records with present records, including such matters as the different values of head losses and values of the coefficient of roughness which are then made available for reference and possible improvement. The actual operating value of such records as these cannot be disputed. As an instance : records may indicate a reduction in quantity of water delivered to the forebay, and/or reduced output from the turbines from other causes ; they may also point to the part or parts where losses occur or trouble lies. Without records of this nature it might be necessary to make fairly exhaustive and perhaps expensive investigations of one or a number of the following : (1) see if the stream flow is not below normal ; ascertain condition ; (2) look for obstructions in the intake

racks, screens, etc. ; (3) look for obstruction and leakage through the conduit and/or a falling in or a deterioration of the conduit lining and/or of the natural side slopes ; (4) look for obstruction in the forebay, particularly the intake racks at the entrance of pipe line ; (5) look for obstruction, excessive friction in and leakage from the pressure pipe line ; (6) look for obstructions in the turbines, which might cause impact losses and the formation of eddies in the

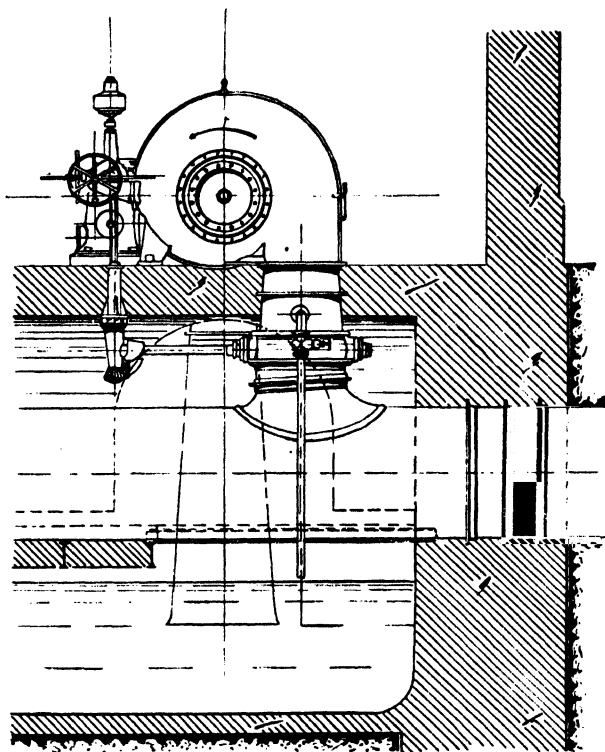


FIG. 23.—Showing setting with straight draft-tube.

water during its passage ; (7) look for obstruction in the tail-race and/or below the draft-tube end.

If the reduction in quantity of water is caused by an obstruction in or above the pressure pipe line, the power and efficiency losses may be proportionally greater because the reduction in flow may result in a loss of head sufficient to affect both power and efficiency. If the records should indicate an increase in quantity of water with a reduction in output, it will usually be found that the cause is due to one or more of the following : (1) Leakage in the pressure pipe line from relief valves, air valves, joints, drain valves, etc. ; (2)

leakage at the turbine connections ; (3) leakage from the turbine casing ; (4) a broken runner vane ; (5) leakage at the runner seals ; (6) leakage in the draft-tube. Usually the trouble will be found in the turbine itself, which, unfortunately, is a part of the power plant generally given the least attention. A reduction in quantity of water of, say, 1 per cent. due to obstruction in the runner may result in a loss in power of at least the same amount as well as an efficiency loss due to the disturbed flow conditions.

Hydro-electric systems are generally located a long distance from load centres and connected to overhead long-distance high-voltage power transmission lines. Hence they are not, broadly speaking, so reliable as steam-power plants situated close to the load centres.

Although water power can never be exhausted by use, because it comes from the total precipitation in the form of rain water, etc., which runs back into the streams and rivers, the amount available *in any one year* has definite limits. This amount varies in quantity from year to year. The amount of water power which can be expected, the total cost of the development, and the possibilities of disposing of the power, etc., all are problems for the engineer. Of all the studies to be made, perhaps the most important one is that of arriving at a good and safe determination of the stream flow and storage for the best and most economical development. Field examinations are always necessary, and in some cases the lack of available information may justify extensive hydrographic and topographic surveys. Unfortunately, the cost of these investigations must usually be kept down to a minimum, which often precludes the making of detailed examinations and surveys. In some countries the engineer can obtain a fair amount of the desired information by consulting Government records, reports, etc.

The principal fundamental requirements for a good project are : (1) Good annual rainfall ; (2) large catchment area ; (3) substantial fall within reasonably short distance ; (4) large areas for impounding flood waters ; (5) good foundation material for a small dam to hold the floods ; (6) low value of land ; (7) reasonably low cost of the development ; (8) adequate market for power within a moderate distance ; (9) reasonably good price for electrical energy ; (10) reasonably low total annual expenditures covering complete operation of the development ; and (11) reasonably cheap terms upon which money is obtained for the development. When a project involves all of the fundamental considerations in the advantageous manner mentioned here, the success of the development is beyond dispute.

In general, developments of the type offering in this country are likely to require a relatively large capital investment, and the return on these investments will depend in no small measure on

how the plants obtain the *greatest number of kW.-hours per cubic foot of water*, and on how the plants are burdened by the terms of and expense of capital for the development, vested interests, costly Parliamentary Bill, etc. For this country, hydro-electric plants will, in general, be supplemented by steam plants, in which case all the load that cannot be supplied by the hydro-electric plants must be obtained by burning fuel.

Putting aside for the moment the question of the most economical development, future progress in the way of storage (excess and ordinary) is certain to attach considerable importance to the saving of fuel and operating expenses, etc., during that part of the year when the stream flow is high and when, perhaps, it would otherwise go to waste ; obviously, the existence of *secondary power* will permit the closing down of an equivalent amount of steam power. Depending on the character of the development, but more particularly on the amount of storage available, secondary power may, in one way, be determined by such a condition as the economic balance between the yearly fixed and operating costs of the additional hydraulic and electrical plant, equipment, etc., and the operating expenses necessary with a steam plant of equivalent capacity. The "base-water" or *primary power* usually means the amount available for 8,760 hours in the year. When storage is used, the estimated available power is more often taken over every hour of every day for 100 to 150 days of the year, the period of time varying with the amount of storage and the probable use of the power, the load factor, the diversity factor, etc. The extent to which storage can be used to augment, equalise, or regulate steam flow depends on a great variety of conditions varying widely in character. In this country the amount of energy that can be delivered in the form of secondary power is likely to be much greater than the amount that can be delivered as primary power. For the larger future developments in this country, most secondary power can be fully utilised only by combination with some other power, such as steam-electric or other hydro-electric plants. In many cases, the *excess* storage of water can be accepted as representing primary power—difficulty will often be found in the determination of the exact dividing line for these two distinct classes of water power. In all cases, the chief question will be the determination of the amount of plant and apparatus that should be installed for a given development using storage, and how much it will cost to develop this class of power in comparison with what is usually taken as the relatively more reliable source—steam-electric power.

It is clear that, failing to obtain a reliable analysis of the rainfall and run-off or the accurate determination of the quantity and

distribution of the stream flow available, it will be impossible to prepare final figures for the most essential matters, such as : the most economical development ; the maximum and uniform power which can be depended upon at all times in all years ; the length of time storage or secondary power will be available ; the actual (*not* the estimated) total energy available corresponding to (*a*) base-water supply, and (*b*) storage. Furthermore, as regards *the economical plant capacity* which depends chiefly on the average annual run-off of stream flow, other important matters are involved, such as : whether the plant is to be operated singly or to be linked up with other hydro-electric or steam-electric plants ; the system load factors and their characteristics ; the anticipated power market (present and future) ; the total development costs ; the estimated profits, etc. All these and other independent and interdependent factors must be given proper study where big powers are concerned.

In the hydro-electric field we have now reached a stage of high efficiency where hydro-electric works and plants can be installed at moderate cost, cheaply maintained, cheaply housed, and cheaply operated. Due to the present state of *high over-all efficiency* of the developed waterways, turbines, generators, transformers, transmission lines, etc., constituting all the main "links" in the developed hydro-electric system (of which the turbine has up to the present received perhaps the most attention, due chiefly to its relatively delayed state of high efficiency), the writer is firmly of the opinion that, now the turbine has reached about the highest efficiency (94 per cent.) that can be expected of it, and that hydro-electric power in great amount has been delivered to the switchboard at 90 per cent. over-all efficiency, the near future will see a more general concentrated attention on those matters affecting improvement in the "*over-all water efficiency*," commencing with the catchment area itself, thence following every movement of the rain water up to and including storage and water supply to the developed waterways. As shown below, present-day maximum efficiency of the developed works, plant and apparatus, etc., clearly indicates that little improvement can be expected in this direction. With proper attention given to conservation of water and head affected by the reduction of hydraulic losses to a minimum, and by refinements in the design of the various essential elements of the development as a whole (starting at the watershed itself), the production of water power is certain to reach a much higher state of perfection, speaking generally, than exists in the world at the present time for the majority of systems.

From a universal viewpoint, very few hydro-electric schemes can claim (*a*) the highest possible practical use of a given quantity of rain water ; (*b*) the best and most economical location of the

development sites ; (c) the most economical development as a whole ; and (d) the very best and most economical system operating conditions. In general (considering the *developed* hydro-electric system) there are to be found several outstanding points in a complete development where efficiency improvement can be effected. The principal points, considered in the order of importance as regards losses, are : (1) Conservation of rain water from the time of rainfall up to and including the point of storage and the entrance through the strainers or screens at the intake of the developed waterways ; also the accurate determination of the economical development with reference to the available water power ; (2) turbines complete ; (3) transmission lines ; (4) feeders ; (5) electric distribution service ; and (6) the open conduit (unlined) and the pipe line.

One of the commonest errors of the present day is in the high value assumed for over-all efficiency of the complete hydro-electric development. Usually the lowest over-all efficiency given is somewhere around 65 per cent., whereas 45 per cent. would—with the majority of existing developments—have been nearer the mark when accounting for all losses from and including the developed waterways up to and including the point where the energy is actually sold, *i.e.*, the customers' service. Neglecting the losses in advance of and above the entrance to the developed waterways, the over-all efficiencies of the *best* and *largest* hydro-electric systems, as also the over-all efficiencies of future similar or larger-sized developments, are given below in estimated form.

Certainly the most important problem of the future will be one of how efficiently water can be utilised rather than one of producing a reliable source of power, which latter has already been overcome. Higher efficiency will come from the study and training of run-off, etc., and from the making of the development as a whole—commencing at the watershed ; that is, by conservation and economical utilisation of the seasonal flow to the very best advantage, and by getting as much of the total precipitation, in the form of run-off water, to the turbines with the *minimum of loss of water and head*. In other words, to deliver the greatest number of kW.-hours out of the smallest total quantity of water, or the greatest number of kW.-hours from the best possible ratio value of run-off to total precipitation.

For the best *modern* and best *future* hydro-electric systems (large systems only), the over-all losses respectively can be expected to come within the prescribed ranges (a) (b) and (a') (b') given in the following :—

TABLE XVII.—SHOWING, FOR THE LARGER HYDRO-ELECTRIC SYSTEMS, THE ESTIMATED AND PROBABLE OVER-ALL LOSSES IN PER CENT. VALUES TO BE EXPECTED FROM THE BEST FUTURE AND MODERN DESIGNS

Hydro-Electric System Losses.*		Per Cent. Values.			
		Best Modern.		Best Future.	
No.		(a)	(b)	(a')	(b')
1.	In the developed waterways, including head-works, short conduit and pipe line	2	4	1.5	2.5
2.	Turbines (main units, exciter units, etc.)	7	10	6	10
3.	Generators and exciters, etc. — —	2	3	1.5	2.5
4.	Step-up transformers — — —	0.8	1.5	0.7	1.0
5.	Transmission lines — — —	6	8	4	6
6.	Step-down transformers — —	0.8	1.5	0.7	1.0
7.	Synchronous machines, substations, and distribution centres	4	8	4	6
8.	Feeder circuits — — — —	5	10	5	9
9.	Low-pressure transformers and distribution service	5	9	4	7
Total — — —		32.6	55	27.4	45

The best over-all efficiency of all the developed waterways from the head-works intake to the turbine shaft (including such rare cases as the late Niagara installation) is :—

Best modern—(a) 91 per cent.

(b) 86 „

Mean value—88.5 per cent.

Future—(a') 92.5 per cent.

(b') 87.5 „

Mean value—90 per cent.

As bulk supply will usually include those items between No. 1 and No. 6 inclusive, the very best over-all efficiency will be :—

Best modern—(a) 81.4 per cent.

(b) 72.0 „

Best mean over-all efficiency—76.7 per cent. (modern).

„ „ „ „ 81.3 „ (future).

Future—(a') 85.6 per cent.

(b') 77.0 „

* Poor hydro-electric systems have losses somewhere in the order of :—

Item.	No. 1.	No. 2.	No. 3.	No. 4.	No. 5.	No. 6.	No. 7.	No. 8.	No. 9.
Losses per cent.	11	18	5.5	2.2	15	2.2	12	16	18

It rarely happens that all these losses occur on any one system, hence they are not additive, nevertheless the respective losses given represent poor design, and they are found in practice. On the other hand (a) and (a') will be of rare occurrence, as they represent the very best efficiencies for each main "link" in the system.

<i>Best modern</i> —(a) 66.4 per cent.	<i>Future</i> —(a') 72.6 per cent.
(b) 45.0 ,,	(b') 55.0 ,,

Two developments are rarely, if ever, exactly alike, hence the limits (*a*) and (*b*), and (*a'*) and (*b'*), are given for what may be called the highest practical range of efficiencies. At the present time the major portion of the world's hydro-electric systems, taken as a whole, have not attained the efficiencies of (*b*); hence the necessity, when making a determination of available power, of allowing for a proper and safe over-all efficiency which, at the present time, is usually rated too high. The above values represent the highest range of efficiencies for good hydro-electric conditions.

So as to be in a position to definitely know the state of efficiency of any subdivision of a water-power plant with a view of maintaining the high state of efficiency as found at the time of its installation and acceptance, it is necessary to have complete information available for reference when carrying out general and/or subdivisional tests on the plant, waterways, etc. General data covering the principal features and dimensions of the entire plant are summarised in the following :—

RESERVOIR—

Name	
Distance from power plant — — — — —	miles.
Area of surface of full reservoir — — — — —	acres.
Area of each foot of depth — — — — —	"
Draft — — — — —	feet.
Capacity — — — — —	cubic feet.
Outlet area — — — — —	square feet.

Surface area of full pond	-	-	-	-	-	square feet.
Elevation of surface of full pond	-	-	-	-	-	feet.
Draft	-	-	-	-	-	"
Capacity with draft of.....ft.	-	-	-	-	-	cubic feet.
Spillway length	-	-	-	-	-	feet.
Elevation of crest of spillway	-	-	-	-	-	"

CONDUIT—

(Closed Conduit)—

Area of intake	—	—	—	—	—	—	—	square feet.
Area of conduits	—	—	—	—	—	—	—	" "
Length of conduit	—	—	—	—	—	—	—	feet.

(Open Conduit)—

Area of intake	—	—	—	—	—	—	—	square feet.
Area of conduit with full discharge	—	—	—	—	—	—	—	" "
Area of conduit with part discharge	—	—	—	—	—	—	—	" "
Wetted perimeter	—	—	—	—	—	—	—	feet.
Length of conduit	—	—	—	—	—	—	—	" "
Depth with full discharge	—	—	—	—	—	—	—	" "
Slope of conduit	—	—	—	—	—	—	—	feet per 1,000 ft.

FOREBAY—

Area with full pond	—	—	—	—	—	—	—	square feet.
Net rack area	—	—	—	—	—	—	—	" "
Net area of intakes	—	—	—	—	—	—	—	" "
Submergence of top of intake with full pond	—	—	—	—	—	—	—	feet.
Elevation of surface with full pond	—	—	—	—	—	—	—	" "
Spillway length	—	—	—	—	—	—	—	" "
Elevation of crest of spillway	—	—	—	—	—	—	—	" "

SURGE TANK (*Standpipe*)—

Distance from turbine	—	—	—	—	—	—	—	feet.
Type (open, tank, differential, etc.)	—	—	—	—	—	—	—	
Net area at pipe-line connection	—	—	—	—	—	—	—	square feet.
Net area at tank	—	—	—	—	—	—	—	" "
Net area at throttle, if any	—	—	—	—	—	—	—	" "
Elevation of connection to pipe line	—	—	—	—	—	—	—	feet.
Elevation of top	—	—	—	—	—	—	—	" "
Elevation of overflow	—	—	—	—	—	—	—	" "
Overflow area with 1 ft. discharge depth	—	—	—	—	—	—	—	square feet.

PIPE LINE (*Penstocks*)—

Inlet area	—	—	—	—	—	—	—	square feet
Net or/and average area of pipe line	—	—	—	—	—	—	—	" "
Length of pipe line	—	—	—	—	—	—	—	feet.
Area at turbine connection	—	—	—	—	—	—	—	" "

HYDRO-ELECTRIC PLANT—

(Open Flume Setting)—

Net rack area per unit	—	—	—	—	—	—	—	square feet.
Intake area	—	—	—	—	—	—	—	" "
Dimensions of flume	—	—	—	—	—	—	—	
Submergence of turbine	—	—	—	—	—	—	—	feet.
Type of turbine	—	—	—	—	—	—	—	

(Encased Settings)—

Type of casing (spiral, cylindrical, etc.)	—	—	—	—	—	—	—	
Area of inlet	—	—	—	—	—	—	—	square feet.
Diameter of casing	—	—	—	—	—	—	—	feet.
Length of casing	—	—	—	—	—	—	—	" "
Area of spiral at several points	—	—	—	—	—	—	—	square feet.
Type of turbine	—	—	—	—	—	—	—	

(Turbine Runners)—

Reaction or impulse	
Kind of buckets	
Material of runner or disc.....	
Inlet of depth of buckets - - - - -	inches.
Total discharge area of buckets - - - - -	square inches.
Nominal diameter of runner - - - - -	inches.
Maximum discharge diameter - - - - -	"

(Turbine)—

Type (vertical, horizontal, impulse or reaction, open-flume or encased, spiral casing or cylindrical, needle or deflecting nozzle, swivel or cylinder gate, single or multiple runner, outside or inside gate mechanism, etc.)	
Rated capacity under.....ft.-head - - - - -	b h.p.
R.p.m. at rated capacity - - - - -	r.p.m.
Maximum gate opening - - - - -	inches.
Maximum net nozzle area - - - - -	square inches.

(Draft-Tube)—

Area at runner discharge - - - - -	square feet.
Area at outlet - - - - -	" "
Maximum gross draft-head - - - - -	feet.
Minimum gross draft-head - - - - -	"
Length of draft-tube - - - - -	"
Area of tube - - - - -	square feet.

(Tail-Race)—

Cross-sectional area at draft-tube - - - - -	square feet.
Depth - - - - -	feet.
Wetted perimeter - - - - -	"
Surface slope - - - - -	feet per 100 ft.

AUXILIARY EQUIPMENT—

(Governor)—

Capacity - - - - -	feet-lbs.
Type (oil-pressure, water-pressure, etc.).....	
Governing system (unit or central)	
Pressure system (open or closed)	
Working pressure - - - - -	lbs.
Closing time (full stroke) - - - - -	seconds.
Opening time (full stroke) - - - - -	"
Difference in speed between no load and full load - - - - -	per cent.
Capacity of oil pump - - - - -	gals. per min.
Volume of pressure tank - - - - -	cubic feet.
Volume of receiving tank - - - - -	" "
Diameters and lengths of piping	inches feet.
Fly-wheel effect of respective parts - - - - -	lb.-ft. ² .
Synchronising device.....	
Load limiting device	

(Pressure Regulator)—

Discharge capacity - - - - -	cub. ft. per sec.
Type (automatic or governor-operated)	

GENERAL DATA

Date of tests

Class of tests (waterways, conduits, pipe line, turbine, etc.)

Tests conducted by

Rated flow of water - - - - - cub. ft. per sec.

Rated power of turbines - - - - - b.h.p.

Rated flow of water, each - - - - - cub. ft. per sec.

Method employed for measuring water

	Plant Test at Full Gate.	Turbine Test at Gate.
Volume of water flowing - -	ft. ³ /sec.	
Leakage of water flowing - -	„	
Net volume of water utilised - -	„	
Weight of water utilised per second - -	lb.....	
Gross head (average) - - -	ft.....	
Effective head (average) - - -	ft.....	
Revolutions per minute (average) -	r.p.m.....	
Theoretical horse-power of water -	t.h.p.....	
Water horse-power - - - -	w.h.p.....	
Brake horse-power (developed br.-test)	b.h.p.....	
Electrical output at generator - -	e.h.p.....	
Electrical output at switchboard - -	kW.....	
Brake horse-power by calculation from generator test - - - -	b.h.p.....	
Method employed for measuring water		
Duration of period covering water measurement - -		hours.
Duration of period covering power measurement - -		„
Plant efficiency - - - - -		per cent.
Turbine-setting efficiency - - - - -		„
Efficiency as rated or guaranteed by makers - - -		„
Water flowing per second for different output (100, 75, 50, 25 per cent. capacity) - - - -		cubic feet.
Head loss (gross head, effective head) for different capacities (100, 75, 50, and 25 per cent.) - - - -		feet.
Brake horse-power developed for different capacities (100, 75, 50, and 25 per cent.) - - - -		b.h.p.
Efficiency at turbine-coupling for different capacities (100, 75, 50, and 25 per cent.) - - - -		per cent.
Efficiency at switchboard or generator for different capacities (100, 75, 50, and 25 per cent.) - - - -		„

Turbine results at various gate openings :—

	Water Ft. ³ /sec.	Brake H.P.	Efficiency Per Cent.
Full gate - - -			
0.9 gate - - -			
0.8 „ - - -			
0.7 „ - - -			
0.6 „ - - -			
0.5 „ - - -			

Draft-tube results:—

Gate Opening.	Water (c.f.s.).	Pressure (Fr.) at (Entrance).	Pressure (Fr.) at (Exit).	Velocity of Flow (f.s.) at (Entrance).	Velocity of Flow (f.s.) at (Exit).	Efficiency in Per Cent.	Relative Efficiency.
.....							
.....							
.....							

Test at load ; ft. head ; r.p.m.

W.h.p. on gross head Per cent. (100).

B.H.P.

Per cent.

Loss in conduit, intakes, and racks	—	—	—	—	—	—	—
Loss in pressure pipe	—	—	—	—	—	—	—
Loss in turbine and setting	—	—	—	—	—	—	—
Loss in tail-race	—	—	—	—	—	—	—
Loss in turbine	—	—	—	—	—	—	—
Loss in draft-tube	—	—	—	—	—	—	—
Loss in difference of level between wheel and tail-water in Pelton wheel or impulse turbine	—	—	—	—	—	—	—

In hydro-electric plants the switchboard may be equipped with devices for measuring and recording the output ; unfortunately, in many cases, little or no attention is given to the input of the plant. In fact, not infrequently the machines have to be put into service without running an efficiency test on them to determine the best loading. When tests are made, curves should be worked up and prints made and posted up in the power house to show the operator the best loading on the turbines. After a turbine has been put into service there are many things that can occur to impair its efficiency. Unless periodic inspections are made and operation checked by efficiency tests, the output of the plant may be seriously impaired without anyone being aware of the conditions. Use should be made of the pressure differential across the pressure supply pipe valve, or a change in cross-section in the pressure supply pipe, as a means of obtaining a record of the *input* to the turbines. Apart from this record, there may be a bad leakage through the turbine gates during the period of shut-down. In fact, it is well known that in many low-head plants the absence of the proper type of power-operated gates makes it impossible for the operator to close and open them properly. There are also water losses which occur around the flashboards, sluice gates, and the water that spills over the dam during the night or other periods, all of which, in most plants, is not accounted for. If we only stop to consider that *one* cusec of water wasted for one year, and for every 100 ft.-head, is the equivalent of a power output which, if sold at only .5d. per kW.-hour, would amount to over £130. Where there is a market for the power, the water above the power house is worth money just as coal is at a steam plant, so why throw one away any more than the other? In a well-operated

steam plant attention is not only given to maintaining high efficiency when the boilers are in service, but also in keeping to a minimum the coal burned during the banking and other periods, so why not take the same care of water so that when the over-all efficiency of the plant is considered, every cusec of water is taken into account ?

DRAFT-HEAD PRACTICE

H_s/d_s	(d_s) Diameter of Tube in Feet.	(H_s) Maximum Elevation in Feet.
2.00	7.0	14.00
2.23	6.5	14.50
2.58	6.0	15.50
3.00	5.5	16.50
3.40	5.0	17.00
4.00	4.5	18.00
4.72	4.0	19.00
5.71	3.5	20.00
7.00	3.0	21.00
9.00	2.5	22.50
12.00	2.0	24.00
17.70	1.5	26.50
27.00	1.0	27.00

CHAPTER IX

AUTOMATICALLY CONTROLLED SMALL WATER-POWER PLANTS

SMALL water-power plants were in operation long before steam power was considered possible. In the early days of water-power development it was necessary, in order that the industries might utilise the water power available, to group them close together where a head-race could be favourably constructed, and generally this took the form of a huddled mass of factory buildings jammed in together as closely as they could be constructed, and the whole works dotted about with shaftings, etc. This class of power afterwards became outgrown, and, when the more economical method of producing power came about in the use of steam power, it was supplemented with *auxiliary* steam plant, which kept growing and improving until the water-power plant was generally used as the auxiliary plant. Then followed perhaps the most remarkable period in the history of power development, a period in which it was shown that power could be generated in one place and transmitted and delivered at high efficiency (over copper wires mounted on insulators) several miles away.

Perhaps the greatest field for *automatic* hydro-electric generating stations is that for the medium-sized and small plants, auxiliary to a larger system. For such plants no operators are required, but a periodic inspection by a line patrolman or any other qualified person is the only attendance considered necessary. Viewed from a technical standpoint, there is no limit to the capacity of generating stations for which automatic control equipment can be designed. In a large manually operated station, however, the cost of operation becomes a small part of the total cost of energy, so the saving by the elimination of an operating staff may not justify the small additional risk involved in the *entire* absence of all manual operation. Furthermore, in large stations a complicated switching system may require operators of good judgment, whose service cannot easily be dispensed with. In medium-sized plants, automatic equipment may be used, which simplifies the operation and protects the apparatus to such an extent that only one watchman with little technical ability is required for general duties.

There is a limit in size, below which it is not economical to develop water power by the ordinary methods with manual operation, due to the cost of operation. While a small station, which cannot afford high-class engineers for its control and operation, may have been fairly successful when it was an isolated system and any trouble thereof only local, now, when tied in to the huge electric distribution systems, a much higher safety factor of operation is necessary for the small station, since any serious trouble may involve the entire system. Therefore, aside from the economy resulting from the saving of the cost of attendance, reliability of operation makes it preferable to control and operate such small stations at a distance, from a large station which can afford the quality of attendance giving the assurance of first-class operation. Moreover, with the present-day development of electrical engineering, it is possible to make the operation of the small station entirely automatic, and thereby take its control entirely out of the hands of the somewhat unskilled class of attendants economically available for such stations, and give it the safety and reliability corresponding to the skill of the electrical engineer controlling its operation.

At the present time the working of small water-power schemes can be classified under three main headings, namely :—

1. Those commonly known as mill sites, which often are hydro-mechanical (even at this stage of the art) and fed from open flumes or from a forebay, but no penstocks, where the mill, factory, or works are built over the river or form part of the hydraulic structural works.

2. Those schemes where the hydro-electric machinery and equipment depends upon regular attendance for control and supervision, such as starting and stopping, and the usual operation. For these schemes any class of electric system may be employed, such as d.c. or a.c.

3. Those schemes where the hydro-electric plant is operated and controlled automatically from one or more distant points, and, as can be done in the case of (2), in order to make the best use of the water power available, they are operated in conjunction with other water-power plants and/or fuel plants (steam, gas, or oil). But these schemes are cheaper than (2) because of the lower cost of supervision and lower cost of plant, switchgear, busbars, cables, wires, etc., all of which generally decide whether a scheme is commercially good or is worth developing or not. Moreover, the generators are more robust and much simpler and the power house in general is simpler and safer. For these schemes (preferably taken in group, but not necessarily so) there would usually be one type of electric system, namely, the a.c. induction-generator system controlled from a

suitable point or points as shown diagrammatically in Figs. 1, 2, and 3.

Definitely setting class (1) aside, there is a choice of two schemes, namely (2) the d.c. or the a.c. synchronous generator system, and (3) the induction generator system (preferably three-phase) automatically controlled. In this, as in every system conveying energy, two fundamental economic questions arise, the best system being that which best satisfies Kelvin's law wherein the total net annual income shall be a maximum or the percentage return on the investment shall be a maximum. Indirectly, this law should ordinarily include security of service, *i.e.*, quality of materials, workmanship, etc. This point, like many more, can be discussed from different angles. In fact, one of the first questions to be asked would be that of relative reliability of the two systems. In some respects each scheme must be taken on its own merits as to general conditions and as regards quality or grade of and conscientiousness of technical help and superintendence, all of which varies greatly. The best that can be done is to put in the best plant, simple and robust in construction, and requiring the least attention; the system which seems to accomplish all these in the best way seems to be the induction generator system.

The relative advantages of system (2) over system (3) will depend on whether there is one water-power plant or a number of plants or possible schemes capable of development within a reasonable area, and therefore the first installation should be chosen with great care and with this end in view with respect to systems in the vicinity. Each water-power scheme, or group of schemes, whether on the same or on different catchment areas (within reasonable distance from each other) must be judged on respective merits, local conditions, relative distance from each other, from the load, and from a fuel plant, and be chosen according to the existing class of electric system and supply (d.c. or a.c., phase, voltage, frequency, etc.), and according to the relative magnitude of available water power, kind and character of load, also with a view to the advantage of diversity by tying-in plants (see Fig. 27) with flow from watersheds of different characteristics and other considerations. In general, the writer believes the advantages would come out in the following order:—

(a) The development of the *smaller* water-power schemes can better be made possible (but often made possible *only*) by one-unit induction-generator stations automatically operated from a suitable distant point; or by self-synchronising synchronous generating stations operated automatically from a distant point.

(b) The development of the larger water-power schemes (coming under the class of small water powers) can be made commercially

(c) For a *single* development of the larger water powers (coming under the class of small water powers), the system to install will depend largely on local conditions, distance from village or town, distance from load and from other sources of power, the existing system or systems in the vicinity, character of load, power factor, whether ample exciting current is already available, and other considerations, such as speed of turbine and/or induction generators, their frequency, voltage, and the distance from the source of exciting current or cost of providing an ample supply.

Granting that the total capital cost (which includes operation) is much less for (3) than for (2), there still remains the question of relative quality of and reliability of power supply. Many years have gone by since the writer last operated this type of plant,* and, like most early experiences, his memory is clear as to the construction, operation, and behaviour of the induction generator on a power transmission system. In those days the question of operation was slightly different to that proposed herein, but the difference was not due to any lack of knowledge of remote control or automatic operation; in fact, the present question is an economic one, the technical problem varying but little, if at all, from that of old.

According to the electrical press there are now in operation in California fifteen automatic or semi-automatic hydro-electric generating stations, having an aggregate output of 20,000 kW. These stations would not have been economically practicable with manual operation. It is now possible to duplicate every operation of manual control and to observe the operating conditions of an automatic station at a distant point, and a system of supervisory control has been worked by means of which, over a single pair of wires, it is possible to "listen in" on a distant station and determine the position of the circuit breakers, water level, load on the station, hot bearings, etc. The present tendency in design is toward the use of induction generators, though most of the stations in operation are equipped with synchronous machines. The economic size limit is placed at 5,000 kW. at present. Amongst the stations at work are three using heads of 750 to 1,150 ft. Nine semi-automatic installations are working on the system of the Southern California Edison Co.—one man being on the premises as watchman and caretaker for starting the plant in the event of a shut-down.

A comparatively large water-power scheme of the early days would now be classified as a small water power. Industrial expansion, coupled with the keen economic necessity existing at present (a condition that is liable to be more acute with advancing years), may in many cases force the development of certain groups

* In 1904, California, U.S.A.

of small water powers now known to exist. With one-unit hydro-electric stations the problem is simplified, and there is a choice of several different systems and different methods of operation. With a group or several hydro-electric stations there seems but one reasonably cheap and commercially possible system. The system proposed is that of automatic operation of stations where the *total* developed hydro-electric power from each river or stream is transmitted to and controlled from a suitable distributing point, just the same as if the respective stations were in separate compartments of one building isolated from each other, and the control board was placed in a different compartment of the same building. To accomplish this in perhaps the most economical manner, the induction-generator system (3) is proposed, and the manner of doing this is shown diagrammatically in Figs. 25, 26, and 27. Of course, we may install synchronous generators and make them self-synchronising by means of damping windings, etc.

Without going into any great detail and without showing the merits and demerits of the various systems, the following points are brought out to show what advantage the choice of system (3) has for this class of water-power development, which class is vastly more numerous in this country than the big water powers :—

- A. Cost of generating station, machinery, and equipment is less.
- B. Cost of operation and maintenance is less.
- C. Much simpler generating station.
- D. More robust and safer generators, as they are more substantial in construction.
- E. Depreciation is less rapid, due to robust and simpler construction.
- F. Less liability to breakdown and burn-outs.
- G. In a one-unit station (which would be the general rule) the only circuit is that from the generator to the switchboard, and then to the line.
- H. There is no collector ring or commutator.
- I. There is no exciter and no exciter circuit.
- J. There are no exciter switches, busbars, nor exciter wiring.
- K. Voltage is generated on the stationary armature.
- L. Revolving part is a robust, solid structure of iron and copper bars.
- M. Generator is safer ; this also applying to the station.
- N. Generator requires no attention ; this also applying to the station.
- O. Generators do not require synchronising apparatus of any sort.
- P. Generators adjust their own phase relations on being switched together. With a one-unit station in particular, there is no real

necessity for governors—in such cases the turbines are designed to give a safe runaway speed, say about 70 per cent. above normal. Also, the turbine capacity is so rated that the generator cannot become very much overloaded.

Q. Reduction in the magnitude of surges. In fact, without excitation the induction generator cannot supply power, voltage, or current. Hence there is always less danger, or a total absence of danger, from heavy current rushes on short circuit such as would occur on the ordinary system.

R. If the circuit breakers open, the fuses blow, or the circuit is opened in any way, the voltage dies away.

S. For a given frequency and terminal voltage the current depends upon the power supplied by the turbine. On the other hand, the frequency of the system is determined by the currents supplied by the synchronous machine (synchronous generator or/and rotary converter); thus there is no necessity for the induction generators to run in synchronism.

T. For a given load the induction generator has a definite speed above synchronism. Ordinarily, the frequency will not change so long as governor-controlled synchronous machines are operated on the system.

U. The generating sets may be automatically started and stopped by remote control, depending on the flow of the river, or they may be started and kept running, etc.

V. In general, the best method of operation will be to drive them without governors, so that their output will be kept constant and the load fluctuation will be taken care of by the synchronous machine. If governors are installed, the initial cost is increased, and, with governor-controlled turbines, the speed of the induction generator will drop slightly with the load. Therefore, in order to divide the load properly, it will be necessary for the speed of the synchronous machine to drop if speed adjustment of the induction generator cannot be automatically and satisfactorily controlled at a distance, but this is not likely, as the governors can be adjusted. Without governor control the induction generators will absorb whatever power is available, speeding up until they slip above synchronism sufficiently to transfer the generated power to the control station, which is the most desirable condition at all times. Conditions and local requirements will finally decide whether governor control is the best or not.

W. The only safety device (neglecting devices for disturbances) would be cut-outs to cut off the set or station in case of accident; this would rarely be possible with low-voltage induction generators.

In the early days there was no station automatically controlled

from a distant point. To run any induction-generator station, some means of excitation must first be provided. Going back twenty or more years we find that several means for excitation were proposed, and that the most promising one since then, and up to the present time, has been that of operating the induction generators upon the same circuit as synchronous generators or rotaries which, under these conditions, automatically adjust their phase relations, this action continuing within limits, with variation of the magnitude of power factor of the load. In starting up an induction-generator plant, the water is let in and the turbo-generator is brought up to about synchronous speed, then the generator is switched in. In the early days this was done through reactance coils to reduce the current rush. After switching in, more water is admitted to the turbine, and the load is divided in this way. In operation, when additional load comes on the system, it first comes on the synchronous machine, and then, as this machine slows down and allows the slip of the induction generator to increase, part of the load is transferred to the latter—the synchronous machine supplying simply the additional lagging current required by the generator when carrying the additional load. Up to the present time one practice has been to allow the synchronous machine to supply at all times all the lagging wattless current in the circuit and, with governor control, the governor on the turbine driving the induction generator will help to stabilise the frequency, the synchronous machine slipping behind the induction generator an amount just sufficient to allow the latter to supply all the power required by the circuit. The frequency of the system will depend upon the characteristics of the external circuit. Actually the synchronous machine sets the frequency, and the power factor of the *total* load is adjusted by the synchronous machine to properly care for the power factor of the induction generators. These machines can have the power factor improved by the installation of electrostatic condensers. Condensers are desirable, but besides condensers, a synchronous machine or rotary converter is required. With governor control the load can be transferred from one induction generator to the other by means of a governor on each turbine, and in this way the load can be distributed at will among the generating sets without any adjustment of the excitation, so necessary with the common system of to-day. In many cases it may be found better to install synchronous generators and to make them self-synchronising than to attempt to improve the circuit conditions by adding condensers or any other type of phase compensators.

In the early days induction-generator installations were of the polyphase type due to the common system then in use, and therefore

little or no difficulty was encountered in obtaining the most desirable requirements, such as speed, voltage, and ample reliable excitation ; the latter was derived from the same station or/and from one or more distant substations in the form of rotary converters, which were usually of the 60-cycle type. In the choice of a system for the available small water powers of to-day, such questions as additional cost for the required amount of exciting current, the amount of available excitation within reasonable distance, the turbine and generator speed, frequency, existing systems, load power factor, etc., will perhaps be more important than the relative advantage of initial total cost in favour of the system proposed herein for group operation of plants. The best requirements call for three-phase supply excited from either rotary-converter * or synchronous-generator stations—steam, gas, oil, or water driven. A large number of stations in this country can supply d.c. or/and a.c. (single-phase) current. In view of the fact that the induction generator is more suitable to high-speed work and that schemes in this country will usually be of the very low-head class, there is likely to exist the disadvantage of low speed of turbine with the expense of turbine speed gear. Perhaps the two weakest points are low power factor (the induction generator being more suitable to high power factor) and distant voltage regulation, *i.e.*, control of excitation from a distant point. The former, with its wattless current, is a disadvantage at any time, but more so here, because the induction generator is unable to supply a magnetising current, and it requires lagging current for magnetisation, which necessitates a larger rotary converter or synchronous machine than would otherwise be required to supply the wattless current necessary for the induction generators as well as the load. As regards the question of operation, if the excitation is cut off entirely, the plant affected is shut down, but with relatively much less disturbance to the system ; however, a shut-down would occur in any case, with any system, for an open circuit or a short-circuited line, hence there is no disadvantage here, only that voltage regulation might not, relatively speaking, be so good as when controlled from the same station. This point is a question of efficiency of operation and is controllable. Nevertheless, to approach fairly well the efficiency of the ordinary synchronous generator, it is desirable that the induction generator shall have high speed, low frequency, and low voltage. With the installation of the self-synchronising synchronous generator, certain points are immediately eliminated from the list of demerits.

Generally, we are in the habit of thinking that regulation of the

* Whereby adjustment of field excitation for the proper d.c. voltage would fix the a.c. voltage.

voltage should be done at the generating station only. Ordinarily, there is a voltage drop along the line from the generating end in the direction of the load end or receiving station, and, generally, the voltage at the latter point must be kept at an approximately fixed value independent of the voltage at the generating end. This being so, for every system in operation to-day,* is it not evident that the operator at the receiving station (or at the control station in the proposed system) is the best person to control, to regulate, and to fix the voltage at his—the load—end † of the system—provided he is able to do so? Moreover, oftentimes it would be desirable for the central station operator to be relieved from certain duties, but rarely the operator at the receiving station. In the proposed system the voltage is controlled at the central control station, which is the point of control for one or a group of automatically operated hydro-electric plants. As pointed out in the last paragraph, should there be a short circuit or a broken line conductor on any of the induction-generator circuits leading from any hydro-electric plant to the control station (see Figs. 25, 26, and 27), then that plant affected would be shut down just as if an accident had occurred on a unit in a generating station containing one or several units, with the exception that little or no electrical disturbance would occur in the former case, whereas in the case of a d.c. or a.c. synchronous generator station the rush of current would usually be sufficiently large to cause a surge, but more likely a shut-down or a general interruption of service if the relays, fuses, or circuit breakers failed to act.

In many places (such as in this country) there are to be found small undeveloped water-power sites located near each other on the same stream or on adjacent streams. In many such cases it is not economical to use the total head of water of the several sites for the development of one large power plant. To develop each site independently, although a comparatively small initial investment is involved, may not be an economical proposition if attendants are required in each plant; but by the installation of automatic control equipment, which eliminates the operating force, each independent site can be developed into an economical plant.

The development and the operation of small water powers follow the same general law of economics as the larger water powers, but there is a lowest limit to the *net* income apart from first cost, or a limit to the minimum percentage return on the investment, which makes it possible or feasible to develop a scheme commercially. It is quite possible that, by means of the system proposed herein (shown

* Such systems as these small plants are more likely to be tied-in with.

† The point where it is definitely known what voltage is desired.

diagrammatically in Figs. 25, 26, and 27), many of the small water powers now generally recognised as commercially poor or impossible of developing, can be made fairly good or quite good business propositions. If the expense of operating staff for three, four, or more *one-unit* hydro-electric stations can be reduced (compared with present conditions) to 20 per cent. of the total cost of operation by adopting the system proposed herein, there is a possibility that more of the small water powers (now considered commercially worthless) will be developed. An assistant from the control station would be required to make periodic inspections, etc., of the automatically operated hydro-electric plants.

The linking-up of several small water powers, mentioned herein, is intended to convey the impression that, if desired, a start can be made with one station only, and, as the energy demand grows, more stations can be developed to supply the demand for energy. The greatest benefit will be obtained where stations are operated in conjunction with *large* * steam stations. Over the ordinary hydro-electric system the method proposed herein shows decided advantages in several different ways; but one of the most important advantages is in the relatively larger proportion of the water power which can be effectively utilised. This advantage alone may, in some cases, decide in its favour, because of the proportionately greater annual saving in fuel costs due to taking the very best possible advantage of seasonal irregularity of stream flow for each and all of the linked-up hydro-electric developments which, singly or jointly, would increase the kW.-hour capacity of the power transmission and supply system. The steam plant might be 20 miles or more distant from the control station. The latter may or may not contain step-up and/or step-down transformers; the former will usually be located in or on the outskirts of a town, whereas the latter is more likely to be located in some remote part of the country. The hydro-electric plant (*a*) of Fig. 26 might be 20 miles or more distant from the hydro-electric plant (*d*). From a commercial viewpoint, the distance is limited (in the ordinary way) by the amount of power available, but with the proposed method of power control and supply, the location of the control station would indirectly allow an increase in the transmission distance from respective water-power sites and would also in this and in other ways tend to make possible certain water powers that would otherwise be commercially worthless.

For many of the largest and most important high-voltage power transmission systems of to-day, the transformers, high-pressure switches, and busbars are located outside the power house—the

* Preferably many times larger in kW. capacity.

apparatus operating under remote control. Surely, as this is a recognised good practice, and as transmission lines are so thoroughly reliable, and as the lines or cables in this case will be very short and the power plant relatively more robust, simpler, and safer to operate, etc., there seems no reason to suppose that anything unusual can happen with the relatively *miniature* system of water-power developments proposed herein.*

There are several distinct classes of automatic control for hydro-electric generating stations: one of these methods is capable of controlling many circuit breakers and is applicable to very elaborate and extensive power systems; another method is for use in smaller power plants. There is also the remotely controlled semi-automatic station in which the generator leads run to the main (control) station where the generator is synchronised by hand, and there is the entirely automatic station where the generator is automatically synchronised into the transmission network, etc.

In the installation of automatic hydro-electric generating stations there is a choice between the characteristics and the cost of two types of generators; *i.e.*, synchronous *versus* induction generators. In a high-head installation requiring a high-speed generator, the induction type will usually cost less than a corresponding synchronous generator with its exciter. On the other hand, in a low-head installation, requiring a slow-speed generator, the induction type may equal or even exceed the cost of a synchronous generator with its exciter. Synchronous generators require direct current for excitation; therefore auxiliary equipment for furnishing the exciting current, and control equipment for the auxiliary apparatus, must be provided for each automatic synchronous generator installation. However, the induction generators are dependent directly upon the systems in which they are connected for their excitation; hence, auxiliary apparatus to furnish or control their excitation is not required. But where systems must supply a heavy lagging current and no spare synchronous generator capacity is available, the addition of an induction generator may not be advisable; on the other hand, where systems have extensive high voltage transmission lines and the power factor is high, the installation of an induction generator may result in a more economical development.

Several of the methods in use for automatic control of hydro-electric generating plants are briefly:—

1. When one of the keys on the control board of a power station

* Ranging from 10 to 20 per cent. of that of the steam-electric plant when considering the installation from the viewpoint of *induction* generators. Of course, an entirely different aspect is presented in the case of the installation of (a) self-synchronising synchronous generators, or (b) D.C. series generators. But this part of the text centres on the squirrel-cage type generator.

is thrown, the corresponding circuit breaker in the distant power station is operated and a signal transmitted back to the operator by the lighting of a small red lamp above the key to show that the desired circuit breaker has operated. A complete control system consists of one manual sending circuit, which is controlled at will by the load dispatcher by the operation of small two-way telephone keys, each key representing one circuit breaker. When the key is thrown in one direction it sends a code of successive impulses which operate the proper selector and close the corresponding circuit breaker. When thrown in the opposite direction it sends a different code to open the same circuit breaker. A large number of these keys can be numbered and mounted in a small cabinet or table. The dispatcher may throw one key at a time, or he may throw as many as he desires. The sending apparatus will select each key that is thrown and send its respective code over the line in proper sequence without interference. Should any key be thrown accidentally and then be restored to its neutral position before having sent a complete code, the sending and receiving circuits will restore themselves to their normal positions without causing any undesirable operation. Should several circuit breakers open simultaneously, there will be no confusion in the sending of the codes respecting the different positions, because the automatic sender will pick out each operation, one after the other, and send them all through without any interference.

2 This system of automatic control is not adopted to either the same or so extensive a power system as the one just mentioned. It is, however, well suited to the operation of hydro-electric generating plants in localities requiring a small but reliable source of power, and may be applied with equal success to the control of small stations. The dispatching system is equipped with a small box or cabinet containing one or more small telephone keys and an automatic telephone dial of the usual type. If the dispatcher is located at the end of the line to the different stations one station is necessary. If he is situated in the centre of radiating lines, having several generating stations on each line, as many telephone keys are provided as there are pairs of radiating telephone lines going out of the dispatcher's office. By means of these keys the telephone dial is switched to any line selected. Suppose that the operator desires to ascertain the water head at station No. 5 on line No. 3. He will close the key on line 3 and with the telephone receiver to his ear will turn the dial to figure 5. Immediately after the dial operation has been completed he will hear five taps of a bell, indicating that he is connected to the desired station. He will then dial the number assigned to water head indication and will hear a series of tone pulses in his receiver, which will indicate the water head.

3. Another method is by means of a drum control, and so arranged that the power lines do not necessarily pass through a manually operated plant, but may be tapped directly into the transmission network. Thus the contacts of a float switch close when the water level in the forebay rises to a predetermined level, thus energising a relay and causing the contactor in the gate-motor circuit to close. The closing of this contactor applies voltage to the gate-motor, which is connected to the waterwheel wicket gate through proper gearing. The motor opens the gate approximately 20 per cent. of full gate opening, which admits sufficient water to start the waterwheel. At 20 per cent. gate opening, the gate-motor contactor is dropped out by the breaking of its coil circuit, which is opened by one of the drum-controller segments, the drum controller being driven through suitable gearing by the gate-motor. The wicket gate stays in the partially open position until the waterwheel comes to nearly synchronous speed, when the contacts, the centrifugal switch, driven from the shaft of the generator, close, thus bridging the break in the drum-controller segment and causing the gate-motor to continue to open the gate. The proper segment on the drum controller then causes the fractional-voltage oil circuit breaker to close, thus connecting the generator to the fractional-voltage transformer taps. A drum-controller segment, thus making contact, causes the generator field contactor to close, the closing of which excites the generator, pulling it into step. By means of controller segments operating on the proper control circuits, the fractional-voltage oil circuit breaker is tripped, and the main line oil circuit breaker is closed. The generator is thus properly connected to the bus and operates at a load corresponding to the head of water available. If the level of the water in the forebay falls to a predetermined minimum, the station is automatically shut down. Where the head of water is constant and full kW. output from the generator is desired at all times, the motor-operated wicket gate may be dispensed with and the waterwheel provided with fixed wicket gates. A motor-operated or hydraulically operated valve can then be used for starting and stopping the station.

4. Another method of control involves the use of a master starting element (such as a float switch), but which may be either a pressure governor, a frequency controlled device, or a relay energised over pilot wires, or any other device which, when operated, will close the circuit of a solenoid or other starting device to admit water to the waterwheel. When the waterwheel comes to approximately 90 per cent. of synchronous speed, the contacts of the centrifugal switch close, which complete the circuit through an auxiliary switch attached to the gate mechanism to close the main line and the generator

field contactors. The closing of the generator field contactor also completes the field circuit of the exciter. Owing to the time lag in the building up of its own field, the exciter excites the generator an instant after closing of the main line contactor, which pulls the generator into step with a minimum disturbance to the system. The generator, thus properly connected to the line, will carry its share of the system load when a governor is used ; but when the gates are motor or hydraulically operated, the load will depend upon the degree of the gate opening. When the master element opens the circuit of the starting device, or when the protective devices wired in series with the contacts of the master element operate, the station will close down. In closing down, the load on the generator is first reduced, and then the main line and field contactors are opened by an auxiliary switch which is opened from the gate mechanism.

5. This, and the preceding method, refer to remotely controlled semi-automatic hydro-electric generating stations, in which the generator leads run to the main station, where the generator is synchronised by hand. Thus, a semi-automatic plant can be controlled from some other station by means of the main power line and two additional conductors for controlling the waterwheel gate opening. The operator first closes the control switch, which causes the waterwheel gate to open, thus starting the generator. The field of the generator is connected to the armature of the direct-connected or belt-driven exciter by the closing of the contactor in the main field circuit, and the generator builds up the voltage as the speed increases. The operator, who has a synchronism indicator across the oil circuit breaker in the hand-controlled station, adjusts the speed and synchronises the generator as if it were a machine in the same station or plant. After the generator is synchronised, the load, which is under the operator's control, may be adjusted to any desired amount. In order to have a lower value of excitation for synchronising than that required for full load operation, resistance is inserted in the generator field circuit. This resistance is automatically short-circuited by a contactor when the two stations are connected. The semi-automatic station, when it is connected to the system, operates at constant excitation, which is practicable because it is in parallel with another station where the voltage is under the control of operators. If the remotely controlled generator is over-excited for the kW. load carried, a leading current is supplied to the system, which improves the power factor, provided that the system load demands a lagging current. This control method may be used when the waterwheel wicket gates are operated by the remotely controlled governor, or by an electrically operated valve which controls a hydraulically operated piston connected to the wicket gates.

6. Also, where the distance between stations is great, the cost of conductors for waterwheel gate control may amount to a considerable sum ; hence, by a scheme slightly different to the above-mentioned (5), these control wires can be omitted, leaving only the main power line between stations. Thus, the operator can start the station by impressing fractional voltage on the line. The fractional voltage may be obtained from a starting compensator or from taps on transformers. In the remotely controlled station, the generator is equipped with an " amortisseur " winding and is started as an induction motor. As the generator comes to synchronism the exciter builds up voltage and excites the generator field, thus pulling the generator into step. Ammeters, which are provided in the control station, indicate when the generator pulls into step, and the operator then opens the fractional-voltage supply and closes the line oil circuit breaker, thus applying full voltage to the remotely controlled generator. The closing of the oil circuit breaker causes a contactor to close, which applies full excitation to the generator and opens the waterwheel gate to full-gate position. To close down the remotely controlled generator, the operator opens the oil circuit breaker in the main station. The generator then speeds up, and the centrifugal switch breaks the coil circuit of a contactor, which drops out and completes a circuit to close the gate.

Example.—

HYDRO-ELECTRIC INDUCTION GENERATOR PLANT

A catchment with a total area of about 15 sq. miles is utilised, and two generating plants are operating under respective gross heads of 68 ft. and 437 ft. The latter is associated with a storage of about 225 days' storage, and the former head is utilised below the first station and in *hydraulic series* with it. The flow from the high-head wheels discharges into a small reservoir of negligible storage, and this tail-water and the water from the lower portion of the catchment area are combined and used at the lower power house. There are in reality two catchments or watersheds, one of which is operated against the other. During spring flows and after rain-storms a quick run-off is received from the lower catchment and is used at the lower-head station. At such times this station does the greatest part of the work, the wheels at the high-head plant being partly shut down, thus saving water in the ample storage above. Under the reverse condition, the lower-head plant supplies a small amount of power, and the high-head station does most of the work, drawing on storage. This stored water when released from the high-head wheels supplies the major portion of water required to run the lower-head station. In the latter case the high-head plant does

about 75 per cent. of the work. To suit the above conditions, the low-head station was developed to about 400 per cent. of its average horse-power capacity.

The high-head station is served by a 7,300-ft. pipe line ranging from 24 to 28 in. in diameter. From the intercepting dam, where it originates, the supply pipe drops gradually for about one-half its length, then rises to within 50 ft. of the elevation of its intake, and then drops away, gradually at first and finally more rapidly where it runs downhill to the power house. At its high mid-point is installed a 10-in. air-vent running up the side of the mountain. From the lower end of the supply pipe a steel header connects through three 10-in. inlets to three 350-h.p. Pelton waterwheels, designed for 410 ft. net head. These units are of the single-runner type with two nozzles each and deflector control. Directly connected to the wheels are three induction generators each rated at 300 h.p., 600 r.p.m. (synchronous speed), three-phase. The rotors are of the squirrel-cage type.

The low-head station contains two 200-h.p. wicket-gate type turbines directly connected to two 200-h.p. induction generators. The low-head plant is fed by a 42-in. continuous supply pipe, 1,650 ft. long.

The length of supply pipe and head involved led to a selection of the Pelton type wheel with deflector hood for use with the induction generators in the high-head station. The question of preventing dangerous racing in case of losing the excitation current influenced the solution of the governing problems. Thus, to guard against a run-away speed of lost excitation a speed-limiting governor acts on the deflector of the water jets, thus limiting the speed to a few per cent. above synchronous value when the line is de-energised. As a result no severe jolts are caused when the line is re-energised. The necessary exciting current is furnished by synchronous steam-turbine units in the main station, which is tied into the synchronous stations by a trunk line. With an induction generator floating on a single circuit, it is possible for the motor to lose its exciting current much more frequently than a synchronous generating unit will lose its total load at a multi-circuit station.

In normal operation the load on the induction generator is the only governor desired, and at any time the load being carried determines its speed, so long as the circuit on which it floats is closed. It has been found that with a synchronous speed of 600 r.p.m. and a full-load speed of 585 r.p.m. the motor will generate its full rating at a little below 615 r.p.m. The synchronous speed of the motor was determined by bringing the motor up to a speed that would move a watt-hour meter disc neither forward nor backward. By increasing

the speed the motor would take on load as a generator in consistent ratio with the increased speed.

The governor closes from full open position in from one and a quarter to one and a half seconds from the time of throwing out the circuit breaker. This means that the run-away speed of the motor dies down to within 2 per cent. of the synchronous speed within ten or fifteen seconds. The governor supplies enough water to the wheel to maintain the motor at a slight overspeed, and thus it is ready to take on load again as soon as the circuit is closed and excitation current restored. A slow opening of the governor follows. The essentials of the governor here are quick action from full to no load and an easy possible adjustment of the governor by hand to vary the speed at which it will operate by 5 or 10 per cent. outside the theoretical speed for which the governor is designed. This is necessitated by frictional changes in the frequency on the line upon which the induction generator is floating and by the need of setting the governor for the speed required to take on loads. The governor includes a dash-pot mechanism which is controlled by a milled wheel. By adjusting this wheel the water can be entirely deflected from the waterwheel buckets, or the speed can be brought up to 10 per cent. above synchronous value.

The lower nozzle of two of the units in the high-head plant is not equipped with a valve, but the upper nozzle of all three units has a needle valve. The lower nozzle of the second unit can be cut off by an 8-in. geared cut-off valve if desired, so that the middle unit can be run at less than 50 per cent. capacity with rated efficiencies. Thus the plant can be operated at any point from zero load to full capacity and units arranged for any load, so that a consistent efficiency can be obtained for various load conditions. The generators are directly connected to the waterwheels by flexible couplings of the pin and leather bushing type which allow the generator to centre itself in its field and also permit the waterwheel runner to centre itself with respect to the water jets.

As regards the operation of these plants, if a motor to be operated as an induction generator is thrown upon the line below synchronous speed, the machine first takes current from the line to come up to synchronous speed as a motor, the motor driving the waterwheel for the time being. Under these conditions there is a rush of current from the line corresponding to the starting of an induction motor under load, and the line is subjected to a temporary surge. On the other hand, if the motor is thrown on the line at exactly synchronous speed or very slightly above it, no effect is evident and no change of speed is audible. When thrown upon the line at considerably above synchronous speed the hum of the generator drops to a much

lower pitch, and if this speed is too high, the circuit breaker will at once release. A no-voltage release switch would be inapplicable, since if the line voltage were lost through opening of the circuit, the breaker would be released and the generator would not take load upon closing of the outside circuit. By equipping the circuit breaker with an overload release or an inverse-time-limit relay, which will withstand the shock of the motor's coming upon the line at slightly above its full-load speed as a generator, satisfactory results are obtained. In this installation, these relays are set for an overload of about 20 per cent. and are designed for a maximum time element of eight seconds, although from two seconds is all that is desired. The increase of power-factor with increase of load is of interest.

The operation of the plants is simple. By opening one of the waterwheel nozzles the set is brought up to synchronous speed (determined merely by sound), and the line switch is then closed. This brings the machine quickly to synchronism. The second nozzle is then opened, over-synchronising the set to an amount required by the load. Considered in more detail, the waterwheel is brought up to synchronous speed by a careful adjustment of the milled wheel on top of the governor which holds the speed at, say, 600 r.p.m. After the machine has taken its load the milled wheel is turned until the governor is entirely backed off the wheel. Then the milled wheel is screwed down to a point where the governor will just take hold and finally back again about three-quarters of a turn. This amount of leeway has been found to be just sufficient at this plant to allow for fluctuations in frequency, and does not cause the deflector to intercept the jet except in case of loss of demagnetising current.

Although provision is made to control and read the load conditions of the low-head station at the high-head plant, in this particular installation, there is no co-ordination of operation between these induction-generator stations and the steam station with which they are connected electrically. The steam-station operator can always ascertain if the induction-generator stations are on the line by the exciting current which they draw. The only way in which the steam station can interfere is through the radial fluctuation in generator speed and corresponding frequencies, which would have to be instantly imitated at the induction-generator stations. Compensation for such action can be gained by the adjustment of the milled governor wheel. In case a transmission line is going to be de-energised the induction-degenerator attendant is notified.

The two induction-generator stations are being successfully operated with two mechanics and a general supervisor who is called in only when necessary.

Example.—

REMOTELY CONTROLLED (SYNCHRONOUS) AUTOMATIC GENERATING STATION

In this installation the generator equipment consists of one 500-kVA., 11,500-volt, 50-cycle, three-phase generator directly connected to a Pelton waterwheel. Since this plant, situated about 3.5 miles from the company's main generating station, utilises the entire flow of the stream, provision is made to bye-pass the water around the waterwheel when it is not running. By use of pilot wires to regulate the opening of the motor-operated needle valves, the operator in the main station has complete control over the load.

As regards details of control of this automatic plant: there are two small transformers which are connected to the bus in the control station step up the voltage from 220 to 2,300 volts for transmission to the automatic plant, where it is again stepped down to 220 volts to operate the bye-pass and needle-valve motors.

In order to start the automatic plant, the operator in the main station pulls a control switch which energises a relay in the automatic plant, thus operating the bye-pass motor in the correct direction to close the bye-pass valve. When the bye-pass valve is partly closed, a limit switch makes contact, closing a circuit which controls the motor, driving a limit switch and its needle valve. The needle valve, which is thus opened, admits water to the waterwheel. By opening a control switch the operator may stop the bye-pass or needle valves in any desired position.

At stand-still and slow speeds the contacts of a centrifugal switch are closed, completing the circuit through a relay to close a contactor. The closing of this contactor short-circuits a resistance, which is wired in the shunt-field circuit of the exciter, thus allowing the exciter to build up voltage. The voltage regulator is set to hold normal voltage. When the voltage reaches approximately normal value the upper contact of a contact-making voltmeter opens, which removes the short-circuit around the coil of the contactor and allows it to close, forming a parallel circuit around the resistance of the contactor. From slightly below to 20 per cent. above normal voltage the arm of the contact-making voltmeter floats between its upper and lower contacts. When the generator reaches 80 per cent. of synchronous speed the contacts of the centrifugal switch open, which gives control of the equipment to the contact-making voltmeter. As soon as the synchronism indicator at the main station shows that the generator is in step, the operator closes an oil circuit breaker which connects the automatic plant to the power transmission system. After the automatic plant is thus synchronised, the load is adjusted

by operating the control switches connected to the control bus and its value determined by ammeters and an indicating wattmeter. The limit switches are so arranged that an overlap occurs in the operation between the bye-pass valve and the first needle valve and the second needle valve. That is, before the bye-pass valve completely closes the first needle valve begins to open, and before the first needle valve completely opens the second needle valve begins to open. The reverse operation is obtained during the closing-down period.

If over-voltage occurs when the automatic plant is not connected to the system, the contacts of the contact-making voltmeter close, dropping out the contactor, which inserts a resistance in the exciter-field circuit, thus limiting the voltage of the generator to a safe value by reducing its excitation. If low voltage occurs, owing to a short-circuit on the system, the contact-making voltmeter operates to drop out the contactor, thus inserting a resistance in the exciter-field circuit, which reduces the generator short-circuit current to approximately normal value. In the case of overheated bearings, thermostat relays installed in the bearings cause the station to shut down. To prevent overspeed, the waterwheel is equipped with deflecting nozzles, which divert the water from the wheel if the speed increases above normal value.

Future Tendencies

Water-power developments will be :—

- (a) The development and automatic operation of small water powers ;
- (b) Unified development of individual streams so as to utilise maximum possibilities ;
- (c) Interconnection of electric systems to permit of profitable interchange of power between plants of different characteristics ;
- (d) Greater over-development than is the rule at present, for the purpose of meeting load factor conditions and a decrease in the number of station units ;
- (e) Selective development and operation of water-power plants of varying characteristics ;
- (f) Substitution of hydro-electric for hydro-mechanical power in all plants ; and
- (g) Provides for greatly increased amounts of storage.

CHAPTER X

REPORTING ON A WATER-POWER PROJECT

USUALLY when Government carries out investigatory work of this nature, several men and/or departments are engaged, and matters in general are very much simplified. The best examples of investigatory work are to be found in Canada and U.S.A. ; the former country is particularly well advanced at the present time. About the close of the Great War, the Government of this country made a start in this direction, temporarily appointing a few consulting engineers (specially experienced in local water-supply studies) to estimate the probable power available as determined from a general knowledge of the characteristics, etc., of the chosen catchments and of the total precipitation thereon, making allowance for a safe total loss factor, and applying a reasonably safe run-off coefficient. Speaking generally, the uncertainty of this method with its involved assumptions, guessing, and probabilities, etc., is obvious to all, particularly when carried out by engineers not familiar with the particular type and character of catchment area. In certain far-off countries where stream gauging has not been carried on or where stream flow measurements have been taken haphazard and are considered unsafe, the independent investigating engineer (not Government Department) usually has a lengthy and sometimes hard problem before him in arriving at, what may be called, a reasonably safe run-off value by this method. In every case where stream flow measurements have been *regularly* taken over a period of years, they should be used in preference to any known method of estimating the probable flow from the relationship of precipitation and run-off, or used in conjunction therewith. Calculations depend primarily not upon the precipitation, although this is the original source, but upon the flow (at the point of diversion) of the particular stream or river under investigation. In making the best use of the relationship method mentioned here, it is well to remember that usually the larger the catchment area the lower will be the unit run-off for a relatively short period of time, that the yield of the larger catchment is more steady and will suffer less drought, that the hourly or daily discharge ratio for maximum to minimum will decrease with increased size of catchment area, that the run-off coefficient of relation

may vary widely for the different gauging stations, and that, as the catchment area and average yield increase, there is usually a decrease in the coefficient of variability, etc.

It may happen that, as the result of only a comparatively short reconnaissance survey, a reasonably accurate estimate can be made of the probable amount of power available, etc.; however, the general conclusions reached by such a relatively rough method is not so reliable nor can it be compared with the results obtained from careful observations of stream flow extending over a period of seven, ten, or more years. In some cases transfer of the information of the flow of one stream is made to that of a closely adjoining stream—this may be of doubtful accuracy and may show great variation.

Isolated or "odd" stream flow measurements are practically useless (sometimes positively dangerous) unless the seasonal time of the year and/or the stage of the stream (at the time the respective stream flow measurements are made) is known. Very careful measurements of stream flow extending over many years' time are usually necessary to enable a really good estimate of the maximum dependable available power to be made, this generally bearing more importance where the development is without storage or pondage features, *i.e.*, for "base" power development only. Measurements of discharge made at the two extreme stages (low flow and flood flow), with mention of seasonal time, etc., are of special importance. Also a more exact knowledge of the available power for *maximum* development, and its relation with the ordinary minimum flow, is decidedly of very great importance.

It is often evident that a water-power project (or a similar one over the same reach) has already been studied, and the reason it has reached its present stage, for financial consideration, is generally because it appears to be a satisfactory one and worth taking up. After making investigations, usually a report is prepared which might or might not be submitted in a very comprehensive manner. A good report would include the following principal requirements: (a) Preliminary reconnaissance-survey; (b) flow, power, and cost estimates; and (c) information of the probable power market. It is often in such a form that a proposed scheme is first made known. However, no matter how well the report might appear to be in this or any form, the common stand taken is that all information, figures, values, etc., originally put forward as to the probable power capable of development, the estimated yield from the catchment and/or from storage or pondage possibilities, the estimated absolute minimum, ordinary and dependable flow at the proposed site, the estimates of costs of total hydraulic works, etc., estimates of the power market capable of development within a reasonable distance and for a good shaped

area, and estimate of total production cost, etc., *all* require checking by an independent engineer or party before acceptance or before money can be "written off" by any financial party. Neglecting design and construction, perhaps the worst blunders of the past have been those due to *over*-estimation of run-off from the catchment, and *under*-estimation of total capital costs and total production costs, and therefore, from this viewpoint only, there will usually exist the necessity for checking a water-power report (requiring an independent investigation). The principal object of an independent report would be that of determining the cost per e.h.p. capable of being developed and delivered, at a competitive price, to a reasonably well-assured market offering a satisfactory load factor. The report (to be checked) would naturally receive more attention and support if accompanied by good maps and photographs and a longitudinal section plan (plan and profile) showing actual survey of the reach (taken from below the power-house site to a point well above the diversion or head-works site), which would indicate the best site or proposed sites for the diversion and/or head-works, the best alignment of the conduit, the best location of the forebay, the best alignment for pressure pipe line, the best power-house site, and the best alignment of tail-race (if of any great length). Any alternative sites or scheme on the reach would be indicated in different colours, dot-dash or distinguishing lines. The report would show all necessary detail sufficiently accurate to enable a check to be made of the existing estimates of *costs* already drawn up for the complete hydro-electric development. If the *main* report is bulky, a valuable appendix accompanying the report would be a summarization of the main report on similar lines to that given at the end of this text. The engineer, chosen by the financiers to check the report, should preferably be a man trained and/or experienced in *all* the various phases of hydro-electric power investigation and analysis, construction and operation.

Briefly, an economical water-power development depends upon the available *flow*, the *head*, the *cost*, and the *power market*. Hence, at least three of the principal conditions to be determined as accurately as possible to bring about the desired economic results are, apart from cost: (1) A reliable hydrometric survey and analysis of the conditions of the stream and storage under investigation; (2) reliable knowledge of the net static head; and (3) reliable estimate of the load factor and character of the load curves. The commercial value of any water power would also depend very much upon the size of, and in general the shape of the power market available for selling energy, the proximity of this market, and also on the initial cost of the total development. To market the output from most

water-power developments (which are usually situated in a remote part of the country), certain favourable rates must be offered which will interest several different classes of possible *bulk power* consumers. Broadly, the classes of consumers directly interested would be :—

(a) Those who contemplate installing steam or other fuel generating stations within commercial distance.

(b) Those industrial establishments and mills now buying energy from competitive undertakings.

(c) Those already operating isolated plants of almost any size, but more particularly the medium and large size of plants.

(d) Those who contemplate using the bulk of energy generated for their own use in chemical or other works, or for replacement of or use as auxiliary power from a municipal or company fuel generating station. Usually, to render economical supply to power consumers and to enter the power market on a proper basis, it is necessary to make a careful study of the available market with the object of gaining definitely reliable information on such matters as the :—

1. Class of power consumers (tramway, industrial, lighting, etc.).

2. Present total costs of operating existing isolated plants and/or class (1) for municipal or company undertaking.

3. Amount of power required.

4. Hours of operation (constant or/and intermittent).

5. Load factor and diversity of maximum loads.

6. Proper and safe basis of respective power and energy sales, to possible big consumers in particular.

7. *Mean* unit selling price of energy at the consumers' terminals.

8. Relative local generation production costs compared with (7), etc.

The future of this country (in particular) offers a very wide field for comprehensive studies along these lines. The first start in this direction may emanate from the recent issue of the North Wales-Cheshire area. Speaking in general terms, this condition for other countries is neither new nor novel, nor would it be advantageous to the industries and public unless existing supply companies are allowed to favourably compete (without restriction) in the mean unit selling price of energy to every possible consumer within the licensed and/or respective areas. The crux of the problem will be *lowest price per unit and best supply of energy free from interruptions* ; the consumer will certainly have little or no further interest in the matter unless it be to his advantage in some financial way, such as that of aiding phase compensation.

Generally speaking, at the present time there are no really good relatively cheap (commercial) water-power projects going begging or

seeking capital. Water power in abundance can, of course, be found in many countries, but it is not commercially good or it is not worth taking up at the moment unless, perhaps, for the express purpose of "cornering" a concession for future profitable use. Where schemes have been studied in a preliminary way, then, assuming the concessions and the cost of land and other rights to be satisfactory and the power market good, one may be assured (from the information usually contained in the text of the water permit) that a hydrological investigation or hydrometric survey is worth while. Usually the first study gone into is that of the stream flow and water power obtainable, as more often mistakes are made in this direction. However, conditions, requirements, and local circumstances usually decide the best method of procedure in the preparation of or checking of a water-power investigation and framing of a report thereon. A new or competing power market and the distance therefrom may in itself decide against a project, and sometimes the power market is the first *and only study* made, because it decides against the scheme.

The scope or extent of water-power investigations differ materially as a result of all manner of conditions, natural and otherwise; moreover, there is a very wide difference in the manner different engineers draw up their reports. Thus, for a given project, one may see fit to omit certain data requiring unnecessary expense and time to ascertain and actually not required for the time being, while another investigator may see fit to elaborate on that very point and go into unnecessary detail at the expense of other matters of greater importance for the moment. This may be due to the engineers concerned having had special or specific kind of training and experience, but more often it is due to lack of knowledge in field investigations and/or to insufficient practical experience in the construction and operation of distinct types of water-power undertakings. Another engineer might consider it necessary (and possibly would if the studies happened to be under Government control and expense) to make a thorough investigation and study so as to involve geology, meteorology, the legal and commercial aspects, etc.; carry out an exhaustive study of the control of the entire main stream showing how the best use can be made of land and water rights, etc.; thoroughly discuss the type of structures for different parts of the hydraulic and electrical works, supports, etc.; discuss the advantage of different sizes, shapes and capacities, etc., and the best type of works, plant, equipment, and auxiliaries; present a detailed analysis, and give mathematical deductions and tabulated results of every important factor entering into the development; discuss the productive, commercial, and administrative details, etc., and have the report accompanied by an

excellent display of maps, and so forth.* With regard to the scope or extent of investigation, its manner of attack, and its magnitude as regards the amount of detail on every point concerned (considered very necessary and sufficient by one but too much or too little by others) there is indeed ample need for engineers concerned to get together and present their views with the idea of standardisation in some way. At the present time it is recognised that in most countries, engineers have no established *set* form, method, classification, or practice in the matter of water-power investigations and/or drawing up of reports. For instance, a water-supply engineer or a qualified hydro-electric engineer might submit a very different report to that of a turbine maker.

Hydro-electric projects vary all the way from a simple scheme to a highly complicated and very extensive scheme. In the one case we may have all necessary stream flow records at hand, a scheme offering cheap land and water rights, and be so concentrated that the entire development and the whole power market are located within the space of a few acres of ground ; the whole scheme financed with " cheap " money. On the other hand, we may have very unreliable, a poor, or a total absence of hydrological records ; a scheme involving large and costly lands and rights ; be so situated as to involve complicated storage features ; offer uncertain foundations for dam or a difficult and expensive diversion structure ; long conduit requiring different and expensive sections such as siphons, aqueducts, tunnels, etc. ; somewhat inaccessible route and bad ground along the route ; expensively built-up forebay site ; long pressure pipe line on a doubtful hillside ; expensive excavations for power house, very inaccessible ; long and costly tail-race ; long transmission lines running in different directions to the prospective power markets ; expensive wayleaves ; uncertain power market in an area already supplied and with prospect of a rather slow growth ; a scheme financed with " dear " money ; vested interests in opposition ; and a scheme burdened at the outset with a costly Parliamentary Bill, etc.† It is therefore obvious that the extent or scope of investigatory work must vary with the character, etc., of the project, and be quite independent of the different views, etc., mentioned in the last paragraph. Nevertheless, a satisfactory water-power report would usually involve the study of such main features as the : (1) Class of report to be prepared ; (2) investigation and analysis of all necessary field information, including absolute reliability of records ; (3) planning of the proposed development along the most

* An independent investigating engineer is usually limited in the extent of investigatory work and analysis by expense and other considerations.

† These do not refer to any one or any particular scheme ; in fact, as a general rule, several of the items mentioned can well be met with in any one scheme.

economical lines, mainly with regard to first cost ; (4) modifications (with comparative costs) due to storage or pondage ; (5) estimates of power available and production costs ; (6) study of power market ; (7) financial considerations, etc. The investigation of the power available and production costs may resolve itself into : (a) A complete compilation of all available rainfall and run-off records applicable to the case ; (b) a study of the reliability of these records ; (c) a determination of the most probable run-off for typical years ; (d) possible modification of flow estimates due to available economical pondage or storage ; (e) estimates of the power available (with and without storage or pondage) ; (f) a study of the available and probable market, primarily to determine and plot the load curves ; (g) estimates of financial returns for *various operating conditions*.

With this information at hand (preferably showing formulæ and methods used) and, bearing in mind that the whole scheme is to be planned along the most economical lines, it might be considered desirable, perhaps very necessary, to prepare estimates in sufficient detail giving : (a) The most economical plant or development capacity ; (b) maximum uniform load which can be depended upon at all times ; (c) length of time that other loads will be available, these loads varying between (b) and the plant capacity ; (d) expected energy output corresponding to (b) and (c) ; (e) financial returns when these conditions are met, etc.

Within the space allowed here, it is not possible to treat in detail with all the different features which enter into or may be required for every class of water-power project and report. Considering the principal technical details only, the body of the *main* report should certainly contain the fullest possible information of the technical features of the whole project, and the report should be accompanied by good up-to-date maps and photographs, giving necessary physical data. The preliminary technical details covered by the *main* report would usually include :—

1. Statement of the catchment and its geology, topography, precipitation, run-off, storage features, and the water supply (absolute, ordinary, and dependable flow with and without storage), etc.

2. Statement of the main storage, giving location of dam, foundations, height and length of dam, type of structure, nature of bottom of and extent of land submerged, storage capacity, etc.

3. Statement as to how the *head* was determined, whether by means of level, contour-map, actual measurements or by aneroid ; stating gross and net head available.

4. Statement dealing with head-works and diversion, giving (a) location and accessibility ; (b) notes on the geologic and topographic features ; (c) type of head-works and diversion ; (d) means

proposed for regulating water supply before its entrance into the developed waterways.

5. Statement as to (a) effect of eliminating (by cutting across, etc.) long tortuous reaches, bends, and so forth, showing financial advantages, etc. ; (b) concrete lining of certain bad reaches of the stream where it is known loss of water occurs or where the coefficient of roughness is excessive ; (c) approximate velocity of flow, near and at the site of diversion for low water and for flood water levels ; (d) nature of stream-bed and banks along the reach but more particularly for a short distance above and at the diversion site.

6. Statement dealing with forebay, giving (a) suitability of site as regards economy and capacity ; (b) probable capacity for pondage and regulating purposes ; (c) accessibility and nature of land submerged ; (d) type of construction proposed and relative amount and cost of rock-cut, excavation, and/or fill ; (e) its location with respect to the pipe line, etc.

7. Statement of pressure pipe line, giving (a) accessibility and suitability of hillside ; (b) number of lines and type of pipe or pipes proposed ; (c) approximate length of different pipe sections, their thicknesses and respective diameters ; (d) total length of pipe line ; (e) character and amount of load for which each pipe line is intended for ; (f) most important details of pipe line, its foundation, backing, anchorage, etc.

8. Statement dealing with power house, giving (a) height above bed of stream ; (b) height above stream water level at low and flood-water gauge heights ; (c) suitability as to nature of ground and its extent ; (d) accessibility of site to good roads and railroad or navigable river ; (e) type of structure ; (f) type and capacity of plant (hydraulic and electrical), equipment, and auxiliaries, etc.

9. Statement dealing with the tail-race, giving (a) probable restricted discharge area in a state of nature ; (b) restricted discharge area anticipated by the location of hydraulic works ; (c) length and type of tail-race from power house to the stream, showing approximate amount and cost of rock cut and excavation.

When all the technical features have been worked out, detailed costs (which should be embodied in the main report) are then drawn up. For instance, assuming we are quite satisfied (from studies originally made) that the specified amount of power really is available, that no unusual engineering difficulties are involved, that the immediate power market looks fairly good, that the cost of all necessary land, expense of buying out vested interests, cost of complete issue of concessions, etc., is reasonable, then, neglecting a knowledge of the terms upon which capital can be obtained (usually a question decided by others, not the investigating engineer), the *capital cost and fixed*

charges for the project should then be drawn up and would ordinarily include :—

(a) Estimated total cost of concessions and land for the entire development, cost of buying out vested interests, etc.

(b) Estimated total cost of surveys, clearing of sites, loss of amenity, etc.

(c) Estimated cost of providing all necessary construction plant, houses, and hutmen for construction staff and men, stores, offices, etc. Cost of making, relocating, and rebuilding roads and railroads to the hydraulic works for transport of materials, plant, etc.

(d) Estimated cost of unwatering ; cost of coffer dam ; cost of construction plant, etc.

(e) Estimated cost of dam, bridge over spillway overflow, scour, channel to permit of water level of reservoir being lowered, gate towers, gates or valves, and operating gear, pipes, screens, racks, etc.

(f) Estimated cost of constructing conduit, including cost of constructing diversion dam and head-works, with spill-weir, gates, screens, etc.

(g) Estimated cost of constructing settling basins, siphons, aqueducts (if any).

(h) Estimated cost of constructing forebay, including cost of allocated plant for concrete mixing, cost of gates, gate-house, racks, etc. Or, estimated cost of constructing surge tank in lieu of, or in addition to, the forebay.

(i) Estimated cost of pressure pipe line with anchor blocks and bolts, supports, bends, control gates or valves, expansion joints, air-inlet valves, Venturi meters, etc.

(j) Estimated cost of generating station building, machine shop, fitting and fixtures, etc.

(k) Estimated cost of hydraulic and electrical plant, apparatus, equipment, auxiliaries, machine tools, piping, etc.

(l) Estimated cost of constructing the tail-race, weir, etc.

(m) Estimated cost of erecting dwelling-houses for the permanent staff and operators, stores, etc.

(n) Estimated cost of legal and engineering charges, expenses of supervision, contingency allowance, and interest allowed during construction period, etc. Including cost of obtaining the authority of a Private Bill.

The *estimated annual costs* would include redemption of capital, also renewal of plant and general working charges in connection with the hydro-electric plant, apparatus, equipment, etc. The allowance for depreciation and maintenance of works would, of course, vary with the type and character of development, civil engineering works and plant, etc. In this text it would be unsafe to give *one set*

of figures to cover every type and class of development. At the best it would be a very rough guide, for the reason that depreciation and maintenance costs depend on such matters as the quality of material used and on the grade of engineering, workmanship, and management; moreover, these costs depend on the length, size, type, character and structure, slope of waterways, velocity of flow, climatic conditions, voltage, factor of safety, etc. The annual costs would also include the salary of staff and wages of all operators, clerical and casual assistance, expense of administration, general stores, oil, waste, and other incidental expenses. It would also include insurance on buildings and plant, etc., and workmen's compensation insurance. For interest on invested capital (allowing for sinking fund charges so as to redeem the capital in a given number of years) the annual installation may be reinvested at a given percentage. In the estimated cost of current per unit there would be included allowances for redemption of capital and, perhaps, payment of dividend on the principal sunk in the development.

Let A = total annual income, and let B = total annual expenditure, then the estimates should show a profit or loss so that conditions satisfy :

$$x = A - B$$

for a *profit*, and

$$y = B - A$$

for a *loss*.

But, where A and B are equal there is no profit or loss.

Expressed in another way, it is evident that when the interest on total capital cost *plus* the total annual cost *plus* the total annual expenses is *less* than the total annual kW.-hours sold *multiplied by* price per kW.-hour of energy sold, then the hydro-electric development shows a profit. But when these conditions are reversed there would be a loss.

The h.p.-year cost of the development, which involves the total quantity of energy delivered and the sale price of energy, can be expressed in the form

$$C_s = \frac{k}{C_e} \left(\frac{K'}{P'} + C_y \right)$$

where

K' = total capital cost.

P' = h.p. year of delivered energy.

k = yearly expenses, made up of interest, etc.

C_y = cost due to energy loss.

C_e = cost of generating the energy per h.p. year. The system cost being expressed :

$$C = C_e \left(\frac{P'}{e} \right) + kK'$$

where e = system efficiency.

One of the most interesting parts of a water-power report is usually the presented physical data in the form of maps and photographs. These maps, plans, and photographs should show catchment areas, contour (isohyets) levels, the site of reservoir, dam, conduit, forebay, pressure pipe line, power house, etc.; in fact, every principal physical link in the proposed development; profiles of non-pressure and pressure conduits should be attached to the report, also a key plan of the whole project. Usually, where the written text of a report is lacking in sufficient detail, etc., good up-to-date maps and photographs help to fill in the gaps; they also help to give a better understanding of the written text contained in the main report. About the most useful are those showing works in full on large-scale contour maps and plans. To show a water-power report off at its best it might be necessary to attach the major part of, or the whole of, the following mentioned :—

1. Plan on the scale of about 1 in. to the mile, showing the catchment, reservoir, dam, head-works, conduit, forebay, pipe line, power house, etc. If there are several catchment areas for the project, or combination of projects, these should be shown in different colours.

2. Plan on the scale of about 6 in. to the mile, showing different parts of the civil engineering works.

3. Plan on the scale of about 25 in. to the mile, showing the dam (in some detail).

- (a) Plan on the scale of about 25 in. to the mile, showing the head-works (in some detail).

- (b) Plan on the scale of about 25 in. to the mile, showing forebay (in some detail).

- (c) Plan on the scale of about 25 in. to the mile, showing power house and tail-race (in some detail).

4. Plan showing the areas of land to be more or less submerged.

5. Several plans indicating, by means of sections, the nature of the ground for the installation of the dam, forebay, or regulating reservoir, conduit, surge tank, pipe line, power house, tail-race, etc. The different specimens of work, etc., can be shown in different colours.

In respect of maps and drawings covering a water-power project, certain Governments request (when an application is made for a permit) that they :—

- (a) Shall be on tracing cloth (linen) cut to uniform size, not smaller than 24 in. by 36 in. and not larger than 28 in. by 40 in., the latter size being preferred, except that lithographic maps may be used for general maps of the project and, when so used, one copy for permanent record shall be mounted on linen.

- (b) Shall have a clear border of $\frac{1}{2}$ in. on three sides, and of $2\frac{1}{2}$ in.

on one of the shorter sides, which shall be the left-hand border of the map.

(c) Shall have a numerical scale and a graphical scale, the latter not less than 6 in. in length.

(d) Shall, if a map, have true and magnetic meridians indicated thereon.

(e) Shall have a space 4 in. by 7 in. on the lower right corner, the upper half of which shall bear the title, scale, etc., and the lower half of which shall be left clear.

(f) Shall, if a map, include public lands, show location of all public land survey lines crossing the project area. If on unsurveyed public lands or reservations, the protractions of township and section lines shall be shown, such protractions whenever available to be those recognised by Government or those officials having jurisdiction over the lands.

(g) Shall be rolled, not folded.

Besides the above-mentioned, the photographs which should accompany a water-power report may involve :—

Views of parts of the catchment area, with a short mention of geology, etc., below each view. Aerial views would perhaps be the most useful—their use need not be considered here.

Views of the proposed reservoir site taken from different points and angles, indicating approximate water levels with lines drawn across the photograph.

Views of the proposed sites of head-works, intake, etc. The views of site of dam or head-works should be taken from up and down stream ends, showing banks in every case. On each photograph indicate the water levels by drawing lines across the photograph.

Views of rapids or falls taken from each side of the stream (at different elevations).

Views of the most difficult parts of the conduit route for those sections of conduit requiring siphons, aqueducts, etc.

Views of the proposed forebay site taken from different angles ; indicate thereon the approximate water level.

Views of the proposed pipe-line route, more particularly the difficult parts of the route. Indicate by arrow-head location of spill-way channel from the forebay and its relation with location of the pipe line. Give a concise statement below each view of the hillside conditions, etc

Views of the proposed power-house site and tail-race, indicating high and low water levels by drawing lines across the photograph. Give statement below each view as to nature of ground, etc.

In addition to the above-mentioned data, etc., a report should, preferably, be accompanied by :—

- (a) Profile of the entire conduit from the intake to the forebay.
- (b) Profile of the entire pressure pipe line from the forebay to the power house.
- (c) Profile of the river or stream (above and below the power reach), indicating estimated height of back-water during floods. For very low-head schemes (where a dam forms part of the hydraulic works) this information is of special importance.

At the present time it is generally recognised that, in describing these investigations, the utmost use should be made of maps and photographs. By their use an experienced engineer can usually extract (without leaving his office) a vast amount of information as to the catchment characteristics, conditions of soil and culture, ground-water levels, possible fluctuation of ground-water levels, geology, precipitation, and run-off conditions, possible engineering difficulties, and/or excessive cost of civil engineering works, etc. Broadly speaking, maps and pictures describing a catchment cannot be too complete, although they may at first sight appear confusing. A good map would show by contour lines the configuration of the surface so that reliable distinguishing lines (indicating boundary surrounding the topographic area under investigation) can be drawn in and the enclosed area planimeted. Tinting or other conventional methods are often used to indicate the character of the ground in different parts of the catchment. The best map would show the contour lines, the positions of rain-gauging stations and other gauging stations with the mean annual rainfall marked in for reference. A desirable map is one indicating forest-covered, bare, cultured, and other areas in the catchment under investigation. Another map (on the same scale) would show, for the same catchment, the underlying geological formations, contour levels, etc., giving the elevation of the rock surface of the different formations, etc.; these contour levels are very useful in the investigation of ground-water, water table, etc., provided the information is reliable.

In conjunction with the maps and photographs mentioned above, the most useful and perhaps most sought-for information is that part of the report contained in *appendix* form, i.e., *summarised statement of the main report*. One such outline or summarised statement of a water-power report is given below :—

SUMMARISED STATEMENT OF THE REPORT

Physical Data

Location.—(Brief statement of location of sites and their respective distance from the nearest tide water, navigable river, railway station, power market, etc.).

General Information :—

River.....	Falls or rapids.....	Accessibility.....	
Catchment area to the intake site	- - - -	square miles.	
Water supply (brief statement) :—			
Local records from 18..... to 19..... show that—			
Minimum daily flow is	- - - -	ft. ³ /sec.	
Minimum monthly flow is	- - - -	"	
Average annual (low year) flow is	- - - -	"	
Estimated available flow with storage	- - - -	"	
Estimated annual rainfall from records	- - - -	inches.	
Storage available	acre-feet	- - - -	square miles.
Area of storage surface	- - - -	- - - -	acres.
Gross head	- - - -	- - - -	feet.
Net head	- - - -	- - - -	"
Possible power at the turbine (assuming.....per	- - - -	- - - -	b.h.p.
cent. over-all efficiency)	- - - -	- - - -	"
Power available for ordinary minimum flow	- - - -	- - - -	"
" " " estimated maximum develop-	- - - -	- - - -	"
ment	- - - -	- - - -	"
Dependable 24-hour power with storage	- - - -	- - - -	"
Output generated	- - - -	- - - -	kW.-hour.
" delivered	- - - -	- - - -	"

Estimate of Cost Complete

A. Preliminary work :—

1. Surveys	- - - -	£
2. Clearing sites	- - - -	£
3. Roads, bridges, railroad, etc.	- - - -	£
		£

B. Storage reservoirs. (Brief statement giving name, location, area, available capacity for each of) :—

1. "a" reservoir.....	
2. "b" reservoir.....	
	£

C. Dam. (Brief statement of length on the crest, height, type, foundations, etc.) :—

1 Estimated total cost	- - - -	£
(a) Cost per billion cubic feet	- - - -	£
(b) Cost per acre-feet	- - - -	£

CA. Weir - - - - - £

D. Conduit :—

1. Surveys and clearing route	- - - -	£
2. Estimated cost for tunnels	- - - -	£
(a) Estimated cost for open-cut (lined)	- - - -	£

E. Forebay. (Statement of approximate area ; cubic contents of masonry or concrete required) - - - - £

EA. Surge tank - - - - - £

F. Pipe line. (Statement of number, sizes, lengths, thicknesses, etc.) - - - - - £

G. Power house. (Substructure) for.....units :—

1. Taking care of water, *i.e.*, cofferdam, filling, face-planking, pumps and pumping, removal of.....cub. yd. of earth, etc. — — — — — £
 2. Excavation of rock (.....cub. yd.) — — — — — £
 3. Concrete.....cub. yd. at.....per cub. yd. — — — — — £
 4. Steel.....lb. at.....per lb. — — — — — £
- £

H. Power-house substructure.....cub. ft. at — — — — — £

I. Power-house plant :—

1. Hydraulic plant (complete) — — — — — £
 2. Electric plant (complete) — — — — — £
 3. Miscellaneous — — — — — £
- £

J. Plant and tools — — — — — £

K. Tail race.....cub. yd. at.....per cub. yd. — — — — — £

L. Right-of-way, etc. :—

1. Reservoirs and flowage — — — — — £
 2. Waterways — — — — — £
 3. Concessions and land — — — — — £
 4. Power transmission — — — — — £
- £

M. Transmission lines : Telephone lines — — — — — £

MA. Substations : Transformers, phase compensators, etc. — — — — — £

N Permanent dwelling houses, etc. — — — — — £

O. General ; Legal : Parliamentary Bill — — — — — £

P. Engineering — — — — — £

Q. Contingencies — — — — — £

R. Working capital — — — — — £

S. Interest during construction — — — — — £

Total — — — — — £

Per cent. value and unit cost in terms of *total net developed horse-power* :—

Unit.	£ per Horse-Power.	Per Cent.
A	£.....	
B	£.....	
C	£.....	
.....		
.....		
Q	£.....	
R	£.....	
S	£.....	
<i>Total cost per h.p., £.....</i>		100 per cent.

1. Operating expenses :—

Superintendent	-	-	-	-	-	£
Operators	-	-	-	-	-	£
Assistants	-	-	-	-	-	£
Labour	-	-	-	-	-	£

2. Supplies - - - - - £

3. Maintenance * and repairs, etc., of—

(a) Dam, tunnels, open conduit, forebay, etc.	£
(b) Storage reservoirs	£
(c) Power house and plant	£
(d) Transmission and telephone systems	£
(e) Substation and equipment, etc.	£

4. Depreciation * :—

(a) Dam at.....per cent. on £.....	£
(b) Tunnels at.....per cent. on £.....	£
(c) Conduit at.....per cent. on £.....	£
(d) Forebay at.....per cent. on £.....	£
(e) Pipe line at.....per cent. on £.....	£
(f) Power-house substructure at..... per cent. on £.....	£
(g) Power-house plant at..... per cent. on £.....	£
(h) Transmission line (copper, insulators, towers, etc.) at.....per cent. <i>respectively</i>	£
(i) Transformers, etc., at..... per cent. on £.....	£
(j) Telephone system at..... per cent. on £.....	£

5. Taxes, insurance, etc. :—

(a) Government water-power tax	-	-	£
(b) Land tax	-	-	£
(c) Income tax	-	-	£
(f) Insurance	-	-	£

6. Administration and general - - - - - £

Total	-	-	£
Interest at..... per cent. on total investment	-	-	£
Yearly cost of operation and fixed charges	-	-	£

* Allowance for *maintenance and depreciation* may be roughly ranged at :— Dam and gates (.5 to 1.5 per cent.); conduit and appurtenant works (3 to 6 per cent.); pipe line (2 to 3 per cent.); buildings (1.0 to 2.5 per cent.); turbines and governors (3 to 7 per cent.); generators and exciters (3 to 6 per cent.); transformers (4 to 10 per cent.); switchgear (4 to 10 per cent.); *overhead transmission lines*: insulators (9 to 16 per cent.); conductor (3 to 9 per cent.); steel towers (4 to 10 per cent.); wooden poles (9 to 12 per cent.); cross arms (3 to 8 per cent.).

Unit cost of power :—

1. Cost per e.h.p. at the switchboard for total e.h.p. is £.....
2. Cost per e.h.p. delivered at for total e.h.p. is £.....
3. Cost per kW.-hour delivered at for total kW.-hour is (pence).
4. Cost per kW.-hour delivered, assuming a load factor of 25 %...(pence).

(a)	”	”	”	”	35	%...	”
(b)	”	”	”	”	50	%...	”
(c)	”	”	”	”	70	%...	”

APPROXIMATE PROPORTIONAL INVESTMENT COSTS

Item of Cost.	Percentage.
Legal advice, preliminary work, and <i>engineering reports</i>	2.0
Lands, etc. — — — — —	7.0
Discounts on bonds — — — — —	9.0
Interest during construction — — — — —	9.0
Legal and insurance — — — — —	2.0
Construction of housing and camps — — — — —	2.0
Construction plant — — — — —	9.0
Cofferdams and tracks — — — — —	7.0
Dams — — — — —	14.0
Power houses — — — — —	12.0
Machinery and apparatus — — — — —	18.0
Head office expenses — — — — —	2.0
Engineering and superintendence — — — — —	5.0
Operators' dwellings, miscellaneous — — — — —	2.0
	100

Note.—Many of these items of cost will vary with the type of development.

CHAPTER XI

WATER LEGISLATION

THE conservation of water supplies is inseparably connected with that of another State resource, namely *forests*. The literature on the influence of forests in relation to rainfall, evaporation, and stream discharge is voluminous, and it is now definitely established, beyond possibility of dispute, that the removal of forests in conjunction with the resulting soil erosion tends to cause permanent streams to become seasonal. Between the years 1843-1883 the Ekaterinoslav Government, Russia, cultivated a forest of 5,000 acres, and established two meteorological stations in this section. Reports show that since the introduction of the forest, showers are much more frequent, and the previously feared dry seasons are things of the past. The stations reported that the average rainfall between 1893-1897 was 18 in. for free lands, while for the forests 22.25 in. It is a well-known fact that the soil of forests retains the water of precipitation more uniformly, and releases it gradually, so that during dry seasons a supply of water is more assured.

Spain has recently been engaged on the investigation of an extensive State scheme of hydro-electric development, involving the construction of central generating stations and the distribution of electrical energy by a network of mains throughout the country. In this project, private enterprise is to be afforded full scope in developing the power available from power reaches of rivers, and supplying electrical energy in bulk to the State mains, the energy from which will be available for distribution to railways, mines, townships, and the requirements of agriculture.

Except in humid countries, where the rainfall is well distributed throughout the year, and where the surface of saturation or water table is near the surface, the proportion of water conveyed in earth channels, which is lost by evaporation and absorption, is a high percentage of the water diverted. In certain soils very little if any proportion of such loss finds its way back to the river of origin. As the aridity of a country increases, and where the rainfall is seasonal, this percentage becomes increasingly high, and it is clear that the use of water for irrigation is carried out at the expense of the power resources of a river.

When water is required both for irrigation and power, perhaps the most ideal conditions which could be obtained to enable water to be available for both uses would be those in which the upper reaches only were suited for power development, and where water is only required on the lower reaches for irrigation. The first condition would be complied with if such a high degree of continuous humidity prevails on the upper reaches that irrigation is unnecessary, and if, in addition, conditions were coupled with steep gradients on the river. The second condition would be fulfilled if the lower reaches only have flat gradients, unsuited for power development, but coupled with such low atmospheric humidity in the region traversed that it is in that region alone that irrigation is essential for agriculture.

The conflict regarding the priority of the various uses of water is one which has assumed considerable proportions in some countries, and most countries have evolved a schedule of uses in order of priority. This order, except for first priority for domestic and municipal uses, must, however, be liable to variation according to the hydrographical and economical conditions prevailing on the river. Owing to the impetus which has been given in recent years to the development of water powers, almost every country has found it necessary to revise its water laws in order to safeguard the public interest, while complying with the demand for concessions, and also to inaugurate hydro-metric surveys to ensure that the State had acquired a knowledge of its extremely valuable water resources, the value of which was never adequately recognised in any country prior to the opening years of the last decade. It is to be noted this recent rush to develop water powers by means of State projects and concessions is not confined to countries like Italy, Spain, Switzerland, Norway, and Sweden, which have to import most of their coal, but it also affects countries richly endowed with coal resources.

Out of a total installed capacity of 1,844,571 h.p. in central electric stations in Canada in 1919, 1,652,661 h.p., or 89.6 per cent., is derived from water. Yukon develops 97.4 per cent. of its primary central electric station energy from water. Ontario develops 95.7 per cent. from water, indicating markedly the commercial adaptability of water power for central station work even where in competition with convenient and reasonably cheap coal supplies. Manitoba develops 95.2 per cent from water, practically entirely from the Winnipeg River power reach. Quebec develops 94.9 per cent. of its central station energy from water. British Columbia develops 89.8 per cent. from water, although first-class coal supplies are available in the province. Alberta develops 43.2 per cent. from water although an abundant coal supply is available. Great Britain has developed less of her available water powers than any other large and civilised

country in the world, with the exception of Russia, published statistics showing that the proportion of her developed water powers to undeveloped ones is 8.3 per cent., as against 43.4 per cent. in Germany and 25.5 per cent. in Switzerland.

The reason for this is undoubtedly, not so much her large coal resources, or the numerous demands on the water for domestic and municipal purposes, great as these are, as the hampering influence of the riparian rights doctrine of English common law. The principles of this doctrine have been abandoned or much modified in practically every other large country in the world, where they formerly prevailed.

Such a multiplicity of opposing interests confront a prospective operator, not only those interests involving the beneficial use of water, but the common law right of a riparian proprietor to have a river flow past his land in practically its natural condition whether he makes use of it or not, that the procedure to acquire a water right by Act of Parliament is such an expensive and uncertain business, that development is held up.

In the review of the Second Interim Report of the Water-Power Resources Committee, the *Water Engineering* (July 1920) says :—

“At present there is no department charged with the duty of exercising general control over the use of water in the interest of the whole community. Notwithstanding the fact that such control in relation to specific purposes, such as water supply, has been recommended as essential by numerous Royal Commissions, Select Committees, and other bodies during the last fifty years, no remedial legislation has been passed. The Committee thinks that such should now be established in relation to all uses of water. The water resources of the country cannot otherwise be properly conserved and fully and systematically used for all purposes, and there is a real danger that at no distant date some of our communities in England and Wales will not be able to provide themselves with proper and adequate water supplies unless such control is established. The provision of adequate and proper public water supplies must, of course, always be the primary consideration, but it is also of great importance that the requirements of the community for water for power and other purposes should be met as far as practicable. The allocation of water has become too serious a matter to be left solely to a succession of Parliamentary Committees which are constituted from time to time to deal with particular Bills and have no continuity of existence, and which are unprovided with machinery by which schemes can be considered in relation to and co-ordinated with natural resources and needs for all purposes.”

The appointment of a permanent Commission, consisting chiefly

of technical men, is recommended by the Water Resources Committee. The primary function of the Commission would be to compile a record of the water resources and water requirements of the country, and for the purpose of obtaining this essential information it is recommended that the Commission should be empowered to investigate and make surveys. It is suggested that in England and Wales all future proposals to take water, whether surface or underground, for any purpose other than for private domestic use, should be submitted to the Commission for their consent.

The principles of English common law in relation to water have never been coded, and are amplified by court decisions given in particular cases, which are often made to apply to cases where the circumstances are totally different from those affecting the case on which the decision was given.

No water right under the common law of England is satisfactorily defined and assured until it has been secured by litigation or Act of Parliament. Uncertainty of tenure militates against development. As the common law is relaxed and State control substituted, so does the beneficial use of water increase.

Common law principles in relation to water are probably less objectionable in humid countries like England, where the water table or surface of saturation is usually near the surface and evaporation coefficients are low, than in tropical and semiarid countries, where most of the water used for irrigation is lost to the river of origin.

The main principles of modern water legislation, which underlie the recent legislation of the Dominions, the Western States of America, Italy, and Spain, and which are really older in their initiation than those of common law, being based on the civil law as established under the Roman Empire, may be stated as follows :—

1. Unequivocal State ownership of all water, both running and standing.
2. The power of the State to alienate the right to interfere with, divert, abstract, and use State owned water by grant of licence.
3. The tenure of the water right, amenity, or patent so granted to be coterminous with beneficial use of the rights granted and compliance with terms of the licence.
4. The domestic and essential irrigation requirements of riparian owners safeguarded.
5. Power of the State to grant compulsory servitudes to operators on, over, or through lands otherwise held for water works, on payment of compensation for the servitude.
6. Power of a high authority to expropriate water rights in the public interest, on payment of compensation.

7. Provision for the consideration of all applications for the grant of water rights by a competent technical independent central authority, with publicity, and opportunities for objectors to file their objections.

8. Power of the authority to grant and record water rights on behalf of the State by licence, permit, or concession, and in accordance with its terms, or to refuse an application.

9. Right of appeal from the decision of the authority to a higher authority or to the courts.

10. Power of the State to expropriate lands and works, and to construct works of public utility on payment of compensation to the owners.

11. Power of the State to alienate the right to develop water powers to companies or others, and authorise the expropriation of lands required for their works on payment of compensation.

12. Provision for the formulation, authorisation, construction, operation, and maintenance, under State authority, of projects involving the beneficial use of water on their own lands by communities of landholders desiring to carry out works jointly, and the power of the State to authorise the raising of money by loan, the striking of rates, to election of trustees by the landholders, the passing of by-laws, etc., for financing and controlling such community projects.

13. Provision for safeguarding the public and local authorities in the construction and operation of water for townships, and supply of water to the public.

14. Provision for safeguarding the public interest by the adequate control by the State of public supply companies and corporations supplying water or energy derived from moving water to the public.

15. Provision for a definite procedure to be followed in the case of all applications for water rights, their issue and record, obligatory both on the applicant and the State.

The underlying principle is that the public grants a water right either free or for a merely nominal charge, and in future years the value of the water right is attached to the land on which it is made use of, and becomes enhanced in value far beyond the anticipations of either the grantor or grantee, apart altogether from the value of the works constructed to make beneficial use of it. When the public which granted the concession or a public supply company requires the use of the water for public purposes, failing legislative enactments, it is the same public which has to pay for this enhancement of value of a right which, in many cases, it had originally granted for nothing.

Insistence on the tenure of the water right and of beneficial use being coterminous does not adequately guard against this danger,

and there are two principal methods adopted for ensuring non-capitalisation of the water right.

One of these methods is by statutory enactment of the principles on which compensation is payable for compulsory expropriation of land and works, stipulating that no compensation is payable for the water right itself, only for the land and works expropriated in accordance with their market value plus additional compensation for compulsory expropriation.

In Australia the duration of a permit does not, according to the water laws, exceed ten years, and, in practice, permits are granted for a term of five years only. In addition to this the permittee has to make a statutory declaration at the end of every year that he has diverted and used water in accordance, and only in accordance, with the terms of his permit.

Many of the earlier administrators in the Dominions and the United States granted perpetual licences, even though such licences were free gifts of the State, without conditions of development. When Government or municipalities desired to operate, the public was required to buy back from the operators something which the same public gave to the operators gratis.

There is much diversity amongst different countries regarding the official or body to whom the duty of granting and recording water rights is delegated by the State. In the Western States of America the power is vested in the State Engineer, an official especially appointed by the Water Laws for the purpose, and there is right of appeal from his decision to a Board of Control, composed of the Division Superintendents under the presidency of the State Engineer acting *ex officio*.

In British Columbia the power is vested in one of the Ministers on the recommendation of the Comptroller of Water Rights. In other parts of Canada it is also vested in one of the Ministers, subject, in the case of Ontario, to certain powers of the Hydro-Electric Commission. In Australia and New Zealand the power is also vested in one of the Ministers, subject, in the case of New South Wales, to certain powers of the Water Conservation and Irrigation Commission, and in the case of New Zealand to certain powers of the County Councils.

The Union of South Africa has adopted a somewhat different procedure to other countries by the establishment of special water courts under a High Court Judge for hearing and adjudicating water claims, for their record, and for granting new water rights.

In some countries in the past, local district water boards have been invested with quasi-judicial powers, and entrusted with the duty of sanctioning water rights. It is believed that in every country

where this has been tried the result has been disastrous, and has saddled future administrators with an almost endless series of puzzles to elucidate, and the courts with a mass of litigation.

Experience indicates that State water must be administered by the State, not by unco-ordinated bodies without technical knowledge. The particular officer of government in whom the power to grant and refuse water rights is vested is the Minister of Public Works in many countries, such as Australia, New Zealand, Italy, Spain, and Switzerland, while in others a special civil engineer is appointed for the purpose, such as the State Engineers of the Western States of America and the Comptroller of Water Rights in British Columbia.

Conservation

For every horse-power delivered to the water turbine-shaft there is saved from 16,500 to 20,000 lbs. of coal per annum, depending on the load factor, size, and efficiency of the steam plant. And, where coal is delivered at the station for £2 per ton (2,000 lbs.), for every 1,000 h.p. output the annual amount paid for coal is £20,000.

Every cubic-foot-second ($\text{ft}^3/\text{sec.}$) of water not used for one year is the equivalent in power output, when working under a 1,000 ft. net head and with an over-all efficiency of 75 per cent., of 85.0 h.p.; this power, when sold at £12 per h.p.-year, is equal to £1 per ft. of net head per cubic-foot-second of water.

CHAPTER XII

HIGH VOLTAGE POWER TRANSMISSION

AT the present time, long-distance extra high voltage power transmission systems are, broadly speaking, non-existent in this country. The principal reasons for the absence of such systems are really fundamental to the country and to the inherent conditions. Due to advances made in the science of electric transmission engineering, steam, and hydraulic power plant and their auxiliaries, transmission lines should be more generally used in the future. The writer contemplates that we are almost on the threshold of a new era of power transmission work which can be brought about by the development of water power; by the concentration of and consolidation of large steam plants; by electrification of railways; and by a more confident outlook and effort in meeting competition at the respective load centres. *Licences* for large areas should foster these changes.

Due mainly to growth and to the more rapid changes and to changing knowledge of the design of power transmission lines, no fixed set of standards has so far been universally adopted. As a matter of fact, manufacturers and designers of importance have no desire (as it were) to "pool" their structural designs, etc. However, there is reason to believe that more attention will in future be directed towards the universal adoption of standards for all lines at the more ordinary voltages. At the present time the transmission voltage range is very wide. In this country, taking 11,000-volt transmission lines as the ordinary low-range limit—using the term "transmission lines" in its broadest sense—the maximum voltage range is nearly 4 to 1, whereas in America (U.S.A.) the maximum voltage range is nearer 20 to 1, or five times greater than the maximum operating voltage in this country. This increase is due to many local reasons not possible in this country. It would seem apparently safe to predict that, within the next ten years, we shall have long-distance overhead power transmission lines in this country operating at 100,000 volts or above. The voltage of to-day is .1,000 times higher than it was forty years ago, *i.e.*, from 220 to 220,000 volts.

One important *future* transmission power problem in this country will be bulk power supply. The change will mean that the character-

istics of the load will be different, hence the load of each electricity supply undertaking should be carefully studied and definitely known, because the bulk power supply company engineers must keep before them complete records and data in order to improve the system load factor by diversity of demand or by actually high load factor service; and he must have a more definite knowledge concerning power factor, load balance, and momentary and sustained peaks. New penalty clauses may come into force with regard to damage that the bulk power customers may suffer if service is interrupted momentarily or for various extended periods. The more important transmission systems are sure to have duplicate transmission lines and/or ring lines, and the line-circuits will invariably be on separate frames. All the more important load centres will, for safety, etc., be fed from two or more directions.*

Transmission lines should be located so that they traverse the less populous territory where their costs, and the penalties of accidents, are less severe. The principal governing factors in the design of the most economical transmission line are: carrying capacity, load factor, distance of transmission, specific cost of energy at the receiving end, and low annual expense on the capital invested. When the transmission line is to be designed simply from the viewpoint of the annual expense of the investment and the electrical energy losses in the transformers, the transmission line, and phase compensators, the total cost will be made up of the arithmetical sum of costs of energy losses, towers, conductors, and earth wire and telephone wires, insulators, transformers, converters (if any), switchgear, land and buildings, wayleaves, phase compensators, and miscellaneous. The cost due to the I^2R losses (for all big lines under reference) will be the predominating factor of the first cost, and its variation will have a direct influence on the total cost. In the design, transmission lines in this country will certainly not be hampered by the consideration of corona.†

A carefully-designed transmission line should have the longest life consistent with reasonable first cost. One of the great dangers in making provision for future possibilities is that; because of the changes from year to year, there is no certainty that twelve years

* The question of electrical limits of power transmission has little practical value in this country, for the reason that, if desired, we can to-day transmit electric power in bulk to distances well over 1,000 miles.

† Some years ago the author was engineer of a transmission system where the line traversed an altitude of 16,500 ft. above sea level. The distance of transmission to the largest substation was approximately 100 miles. Due to climatic conditions, altitude, and other considerations, the economic design of this line was limited by corona; the ratio of volts/miles had to be kept at approximately 500, due principally to the effect of corona.

hence, or even six years hence, we shall want to install the same design as to-day. The layout of the transmission system is of prime importance in ensuring a high degree of continuity of service, and in this regard future installations should follow the leading features of the best and most economical line location and correct line engineering. The development of many of the future large power transmission systems will, due to unalterable circumstances, virtually be a compromise between what is desired and what future circumstances demand and permit. Obviously, a correct decision of the type of transmission line, character of supports, total conductor, and earth wire capacity, etc., which will provide the highest *ultimate* economy as well as excellence of/ or the best service, is perhaps impossible. The designer must, of course, determine as accurately as possible the most economical voltage, the most economical span length between the towers, the most economical size of conductors, and the most economical phase compensation, and the best he can do is to use good judgment (based on a broad and sound practical experience) in estimating future developments of this character.

Great difficulty may be encountered in this country in obtaining easements covering the location of towers and suspension of conductors and wires operating at very high voltages. So far, there seems to be no general standard by which the width of a right-of-way may be determined. Where two power lines are to be located over one right-of-way, it is desirable to have the lines far enough apart so that the towers of one line will not strike the other line on falling. In cases where it is important to use the right-of-way in the most economical manner, it would be best to use two-circuit towers having the conductors of each circuit arranged in a vertical plane on one side of the towers; by a proper choice of phases, this arrangement permits of a lower line reactance. On the other hand, a maximum width of right-of-way is required when conductors are arranged in a horizontal plane. In computing the side-swing, allowance must be made for swing of the suspension insulators. Side-swing of conductors should therefore be based on the longest span ordinarily used, and the extra width should be secured wherever exceptionally long spans are made.

Line supports, for "first-class" installations, consist of rigid (wide base) structural steel towers, the square and tripod latticed structural steel poles, the tubular steel pole, the concrete pole, and the semi-flexible pole and tower. With future years much less use will be made of wooden poles and structures on all first-class installations. With long-distance transmission lines carrying large amounts of electric power, the voltage of transmission is necessarily high and the size of conductors large; therefore, sufficiently strong and high structures are required for the line supports. Experience proves

that structural steel is the most suitable and, in general, the most economical class of material to use for e.h.t. line construction.

As yet there is no recognised standard between the span and sag and the separation between conductors. Because of the high stress at which the conductors may have to work to obtain the highest ground clearance and longest possible span with a given height of towers, specifications should be very rigid. The conductors must be spaced so as to provide sufficient clearance between adjacent conductors and between their supports. The most economical span of towers is defined by the cost of towers and insulators. The weight of a steel tower is very much dependent on the tension of conductors either in unbalance or in the deflection of the line. The failure of steel towers has almost invariably been caused by the buckling of main compression members. The best practice prohibits the use of castings in the main structure of transmission line supports.

There is still considerable difference of opinion as to what should be taken as the maximum loading in designing a transmission line, and designers are at variance as to a proper estimate of the unbalanced tension. Two widely different conditions are assumed, namely: (a) that line supports should have a maximum possible unbalance in tension when the conductors are totally broken down on one side; (b) that line supports should have a small unbalance when a single conductor is broken on one side. Past experience proves that conductors very rarely break off totally on one side. It seems that (a) is too much on the heavy side, requiring thicker steel, while (b) leans a little on the lighter weight or thinner section of steel structure. As a compromise, it is possible that future designers will allow for one of (b) for every six or more of (a). The (b) type may take the place of dead-end or angle towers. In working out the design of towers for extra high voltage trunk lines, a reasonable basis is to assume that sleet will form $\frac{1}{2}$ in. thick and weigh $\frac{1}{2}$ lb. per ft. run at the windage of 17 lbs. per sq. ft. of normal plane area. Experience indicates that *heavy* sleet will not form when the wind pressure is high. In determining the mechanical factor of safety, it is necessary to assume certain properties of the factor as safeguarding each of the possible elements of danger, such as the error of design, workmanship, excess loads, deterioration of materials, and other considerations.

The weight of line insulators is dependent on the transmission voltage. In future transmission development the suspension type of insulator will come into greater use. Up to the present time the pin type of insulator is used in this country exclusively, and its use will continue for voltages up to about 66,000 volts, beyond which the suspension type of insulator will be exclusively used. In special

cases, where a higher factor of safety is required, the suspension type of insulator may be used for very much lower voltages than 66,000 volts, but the relative cost will be greater. When considering the most economical span, the insulators tend to make it longer than the most economical span for the towers alone. The rated voltage principally defines the cost of insulators, while the cost is mainly dependent on the weight of insulators. For a given line, a wider right-of-way, a higher tower, and a longer cross-arm is required for the suspension type than the pin type of insulator. The cost of a suspension insulator for a given voltage * is dependent on the number and the size of units, and is a minimum for a certain number and size. The writer points out these important considerations because they are applicable to designs yet to come into ordinary practice in this country.

Future long-distance transmission lines will universally operate at the highest commercial or permissible voltage. In this respect it is interesting at the moment to note in the technical press the relative merits of the lately installed 220,000-volt † system, and, incidentally, the writer would like to remark that twelve years ago the operation within ten years of such a transmission voltage was predicted.

The near future is likely to see a change in the empirical formula, expressing the most economical transmission voltage, for the reason that it omits the important factor, *specific cost of energy*. The amount and cost of power to be transmitted is a very important factor in determining the economical transmission voltage.

In the writer's opinion the transmission system and voltages in common use—without in any way neglecting the introduction of new systems, such as d.c.-a.c. converter systems—will be the three-phase, high-pressure transmission from generating stations to substations or to other generating stations, or to both, for interconnection of power systems, at voltages of 20,000 to 100,000 volts or above, and three-phase intermediate distribution from generating stations to substations or generating stations, or both, for interconnection of power stations at about 6,000 to 20,000 volts. The above-mentioned transmission lines may be run for short or long distances, and may be tapped at any desired point. In the distant

* The suspension insulator string should preferably be graded and shielded in such a way as to increase the insulation and decrease the voltage stress on those units of the string near the line-conductor, so that these units can absorb the excess voltages due to switching, etc.

† See *Journal Inst.E.E.*, vols. 46 and 47, 1911. On p. 167, vol. 47, the writer stated: "I do not hesitate to predict that during the next ten years transmission lines will be operated at above 200,000 volts, but as to the ultimate voltage for practical operation of our lines I am unable to express an opinion."

future we may see the industrial parts of this country connected with a large network of interconnecting transmission lines between load centres, and these load centres may be supplied from local and super-steam stations and steam stations at the mines, and hydro-

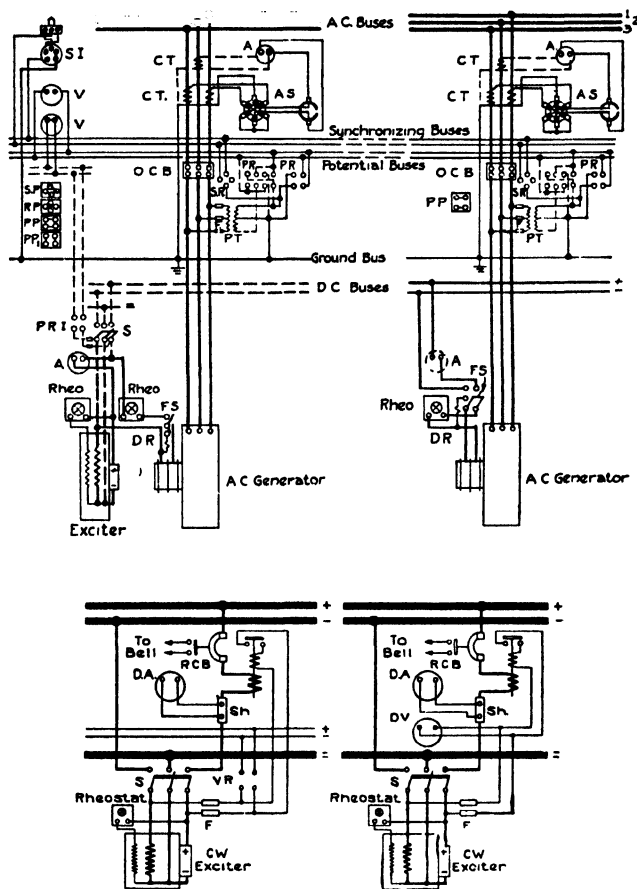


FIG. 28.—Showing wiring connections of the three-phase generator and exciter circuits.

electric stations from sites situated both very far and relatively near the said load centres. The continuity of service will in consequence be well assured, and the necessity for spare plant will be greatly reduced, perhaps far beyond what can be obtained from any source of power supply in existence here to-day. The successful operation of these future interconnected transmission systems will require

complete co-operation of load dispatchers in control of the system operations.

It is good policy to install the higher-voltage system, as this system will, in the course of time, justify its adoption by reason of future growth not always apparent at the time the choice is made. A fairly common mistake in the past has been under-estimation of the ultimate demand for power and the consequent adoption of far too low a transmission voltage.

Looking well into the distant future—with particular reference to certain of the Colonies—the mechanical elements of design with the *very high* voltages may involve structural strains of a magnitude not hitherto encountered. The insulation and the separation of conductors may offer new features; the voltage regulation may require large equipment of an unusual type. The total losses on the line may make up a load on open-circuit sufficient to burn out, due to overload, any one or perhaps more main generators, group of transformers, or converters, unless special conditions such as reversed field excitation, etc., are used.

As the total cost of transmission is practically inversely proportional to the power factor of transmission, with the very large future systems more attention than is now given to present-day electric power systems must be devoted to the matter of the most economical phase compensation. Obviously, phase compensation is economical when the efficiency of transmission is improved and the annual expense of the system per unit of energy is decreased thereby. Large transmission systems such as we shall have in future may not be able to give satisfactory service without synchronous machines (condensers), and voltage control may not be accomplished satisfactorily at the generator alone. For successful operation of the lines anticipated, it may be necessary to have practically a uniform voltage along the line not only for one load, but for all loads. By maintaining constant voltage along the line, troubles, such as voltage rise due to dropping loads and insulation strain, will be reduced. Synchronous condensers (distributed along the line) will have the tendency to act as automatic magnetic brakes against increases in voltage. In all cases, voltage regulation is greatly impaired by low power factor, and the efficiency of a transmission line will depend on the amount and power factor of the load. When the power factor is not very low, the advantage of installing synchronous condensers, for lines *not* carrying large amounts of power, may be doubtful. The installation of synchronous condensers not only requires a certain amount of investment which is high, but the machines and transformers also consume energy to operate them. Moreover, the total energy consumed has to be transmitted over the line.

For certain lines and localities the use of earth wires should be encouraged, as they are effective and beneficial in certain countries, regions, and localities, and for certain transmission line installations. Also, their use is effective in reducing the higher induced voltages sufficiently to bring them close to the value which high voltage apparatus and line and transformer and switch insulators can safely withstand. The *proper location* of the overhead earth wires is

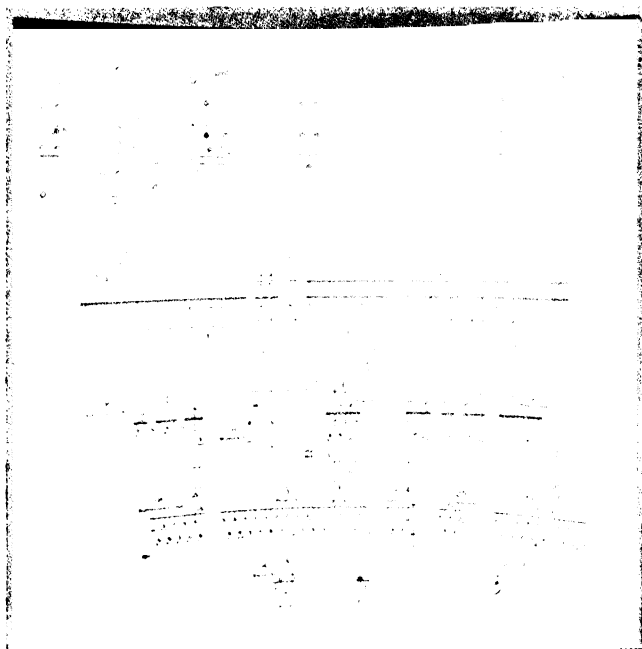


FIG. 29.—Showing a dispatching board which controls two electric supply systems.

decidedly above the power circuits, and not below them. Steel-cored copper should be used, as it is strong and does not rust.

Lightning, switching, and other phenomena cause disturbances on the line conductors. Lightning may affect all the line circuits simultaneously, but if circuits have different routes, simultaneous lightning troubles are not likely. In the opinion of the author, the proper value of a section of iron wire as line conductor installed at the terminal ends of extra high voltage power lines, as a protection against lightning and short-circuit current, is not properly understood as it should be. By virtue of the higher reactance of such sections of a line it might be possible in some cases to obtain sufficient reactance

without the addition of current-limiting reactance applied in other ways. This is a question requiring investigation (see also p. 219).

For long spans, where the strength of hard-drawn copper is not sufficient, the copper-clad steel wire is now finding a ready application, and, in certain instances, the steel-cored aluminium cable should be used. At the present time copper-clad conductor is on the market which has a strength 150 per cent. greater than copper conductor. The limiting of short circuits is desirable where the *all* aluminium conductor is used. Skin effect is quite appreciable in aluminium and copper cable with a steel core. Where conductors are supported on the same plane or in vertical order, the line current may be considerably unbalanced under no-load conditions. At the present time there is no recognised standard or basis for conductor calculations, with particular reference to the *maximum* and *average* values of power, voltage, and power factor, etc. In the calculation of voltage loss for all the overhead extra high voltage (a.c.) power lines now operating in this country, the simple impedance method, which neglects the electrostatic capacity of the line, may be used. It will give sufficiently accurate results for all practical purposes. The accuracy will depend largely upon the length and voltage of line, frequency, and power factor. The total impedance drop should always be distinguished from the voltage drop. In the case of long lines (over and above 50 miles) the methods likely to come into general use are the different forms of hyperbolic functions and the convergent series. Many different graphical solutions for voltage drop and power loss are sure to be employed.*

A transmission line is subject to damage maliciously done or due to accident, but, neglecting these, it is thought that, at the present time, the weakest link in the complete transmission system is the insulation both at the line end and on extra high voltage transformers and switches.

Many years have gone by since the writer first recommended the delta-star connection (star on the transmission-line side) with neutral point earthed direct or through resistance as the best all-round transformer connection; this practice is still recommended as the best for all large power transmission systems.

With respect to earthing the neutral of three-phase transmission systems, these divide themselves into two main classes, each of which has its own particular characteristics radically different from those of the other. The two classes are: (*a*) those systems transmitting at generated voltage; (*b*) those systems transforming and transmitting at higher voltages.

* See "Electrical Design of Power Transmission Lines," by W. T. Taylor and R. E. Neale (Chapman & Hall).

For the former systems, analysis leaves no doubt that the preponderance of opinion is in favour of operating with the neutral earthed through resistance. This generalisation is to be understood to apply only where the generated voltage is used on the distribution system. The experience of many power companies indicates that: (a) the tendency is towards operation with some form of earthed neutral; (b) earthing the neutral through a resistance, in preference to the "dead" earth; (c) the relation between voltage and value of neutral resistance as indicated by calculation of the "volts per ohm" shows that a fairly consistent policy has been followed in proportioning the neutral resistance for the various voltages; (d) where earth relays are used the neutral resistance is generally higher than for systems relying on phase relays; (e) the cast-iron grid resistance (mounted on insulators) seems to give the best and most satisfactory service.

In the past it was found that "dead" earthed systems operated quite successfully for a time, but it became evident with the increase of generating capacity that such earths and short circuits as occurred began to cause disturbances of increasing severity. It is known that the first step taken to minimise the effects of the severe short circuits was to place reactors first in the generator leads and then in the outgoing feeders. Eventually, however, the increasing severity of the disturbances caused by earths led to the insertion of resistance in the neutral connection.

The value of the earth resistance must be adjusted in accordance with: (a) the voltage; (b) generating capacity; (c) number of points at which the system is earthed; (d) system of relays; (e) size of reactors used; and (f) electrostatic capacity of transmission (overhead and cable) lines, etc. At the present time it appears to be a well-established practice to earth at each generating station. The majority of systems find it advisable to earth only one generator on each section of bus. Obviously, the number of points at which it is found necessary to earth will be dependent on the size and extent of the system and to the transformer connections used.

There are several forms of relay protection in use, and all seem to give more or less satisfactory service. The systems of most importance are: (a) earth relay operated from a sheath type (series) current transformer on each feeder; (b) earth relay inserted in the common connection of three current (series) transformers; (c) a similar system operated in connection with a differential relay interconnected with a current (series) transformer in the earth neutral. It is worthy of note that the sheath-type transformer is not affected by such triple harmonic currents as may be induced in the common connection of current (series) transformers when a heavy load occurs

on the feeders, due to slight difference in the transformers, and consequently may be adjusted at a lower setting.

For systems transmitting at a higher voltage than generated voltage there exist several well-defined reasons for the prevalence of the "dead" or *solid* earthed neutral. Some of the reasons are : (a) higher transmission voltages where a "dead" earth stabilises operating conditions and simplifies the insulation problem ; (b) lower maximum currents to earth both on account of the higher reactance in all parts of the system, especially the step-up transformers, and of small generating capacities on the average ; (c) greater use of outdoor installations at the higher voltages where line damage is possible from heavy earth currents. The limitation of the "dead" earthed neutral is of course the maximum short circuit developed in case of an earthed phase and the damage which may result therefrom. On systems with "dead" earthed neutrals the usual practice is to earth at fairly frequent intervals. On long transmission lines at very high voltage both transmitting and receiving ends are almost invariably earthed, while in high or medium voltage systems the neutrals of the transformers are frequently earthed ; not only at the generating stations but also at all or nearly all step-down systems. This practice is undoubtedly due in part to the simplicity of making "dead" earth connections.

The only argument in favour of operating a system with unearthed neutral is the possibility of continuing in operation with one phase earthed. In the early days of power transmission, when distances were small and voltages low, this was possible not infrequently, and on systems in undeveloped territory, especially with lines running radially out from a central power station, such operation became of distinct economic advantage. However, as the size of these systems increased it was found that in the majority of cases the increased charging currents on an earthed phase not only made continued operation impossible, but developed excessive voltages at other points of the system, which resulted in one or more breakdowns, thus causing serious interruptions to considerable portions of the system as well as material damage to valuable apparatus and equipment, etc.

The earthed neutral of a transmission system performs two definite functions, namely : (1) it acts as an anchor to hold the generator windings, transformer windings, and line wires at potentials to earth which are within the values for which a system is designed ; (2) it completes, in case of an earthed phase, a closed circuit through the generator which short circuits the electrostatic capacitance of a system to earth and prevents the building up of arcing earths involving disastrous voltages.

The "dead" earthed neutral limits the voltage stresses that may

occur at normal frequency to a value of approximately 50 per cent. of that which may occur with any other operating arrangement. The creation of excessive voltages in transmission systems, both from internal surges and arcing earths, depends largely upon the amount of energy stored in electrostatic capacity of a system which increases directly with the distance of transmission and as the square of the voltage. Hence the increasing distance and increasing voltage of transmission systems have been the definite factors leading to earthing of the neutral.

Four factors must be considered when designing a neutral earth for transmission systems, namely: (1) excessive voltages must be taken care of; (2) excessive currents must be prevented; (3) due consideration must be given to conditions imposed by selective relaying; (4) due consideration must be given to reducing to a minimum the effect of system disturbances. Operating experiences show that dangerous voltage disturbances will become more and more prevalent as the distance and voltage of systems increase, unless suitable earthing of the neutral is resorted to. At the time the writer was engaged—twenty years ago—with the California Gas and Electric Corporation (now under a different name), the 60,000-volt lines from each hydro-electric station to the load centre had the neutral in each case solidly earthed at respective sending stations. Various attempts were made, and conditions studied, with resistance in series with the line and earth, and with earth connections at receiving ends.

From recent statistics it is evident that there is predominance of the "dead" earthed neutral on the higher voltage systems. The reason for this seems clear when it is remembered that the reactance of a system goes up at least in proportion to the voltage, with the result that relatively small currents flow in case of an earthed phase at the higher voltages. From the point of view of relay operation, the "dead" earthed neutral is the most satisfactory arrangement, since it allows ample currents for positive selective action of relays and permits using the relays for protection against earths and short circuits. Nevertheless, in the last few years there has been a fairly marked trend toward the use of higher resistance in earthed neutrals, especially in the higher voltage systems. Insertion of resistance in the neutral reduces the distortion to the voltage triangle in case of earth, as well as the average drop in voltage. Prevailing practice calls for earthing resistances made up of cast-iron grids for either in-door or out-door operation. The most recent development in methods of earthing of neutrals is the use of a reactance instead of resistance. If for any given system the reactance is properly proportioned it will exactly neutralise the capacitance of the trans-

mission system to earth, so that in case of an earthed phase no current will flow through the fault. It is now claimed that arcing earths are absolutely prevented by this device.

The present situation as to earthing the neutral of transmission systems can be summarised as follows :—

1. Earthing the neutral has proved successful to a very high degree in limiting trouble from excessive voltage on transmission systems.

2. The trend of present practice is definitely toward earthing the neutral of large transmission systems.

3. Increasing distance and voltage have been the factors necessitating this practice.

4. On systems transmitting at generated voltage, the neutral usually is earthed at each generating station with resistance of low value.

5. On systems transmitting at higher than generated voltage the neutrals of the transformers are earthed at each generating station, and in some cases at some stations and in other cases at all substations.

6. Where resistances are used they are generally made up of cast-iron grids with time ratings at maximum current of thirty to sixty seconds.

7. The use of arc suppressors has been very generally abandoned. The high interrupting capacity breakers and selective relays are found to be a better solution.

When designing a transmission line the object in view is perhaps the most economical transmission line. In practice, the actual results, more often than not, vary within a wide range from *the most economical* transmission line. The cause may be due to (a) an imperfect knowledge of this class of civil, of mechanical, and of electrical engineering; (b) lack of practical experience; (c) want of proper understanding of *all* the present and future independent and interdependent variable factors, etc.—in fact, the essential requirement is to be certain that every item entering into the problem of *the* most economical transmission line is receiving proper attention before any further step is taken. This is very often where the first and most important discrepancy is made, and perhaps, added to it there may appear every evidence of an inaccurate or an unsafe interpretation or an improper application of one or more of the varied and rather complex economic problems, technical details, etc. In the design of the most economical transmission line a number of assumptions and probabilities must first be made, some or all of which may offer wide and varying degrees of discrepancies in the calculations upon which the transmission line is to be designed.

Moreover, seeing that the transmission line itself is but a part of the complete electric power system, the numerous economic problems involved may vary widely and be seen to require the solution of many variable factors, difficult and sometimes impossible to solve to anything like a fine degree of accuracy.

As regards the line conductor, designers do not take very kindly to the two following statements :—

1. They have not designed and constructed the most economical transmission line because of the uncertainty of basic assumptions, etc. ; and

2. That extreme refinement in calculations relating to the determination of the *proper* size of conductor, usually represents an unnecessary expenditure of thought and labour, in view of the fact that the final choice will in any case fall between two different standard sizes of conductors differing appreciably in area, one or the other of which must be selected for commercial reasons quite independent of the *exact* size which theoretical calculation would show to be most advantageous.

Obviously, unless the designer is fully capable of solving, within a fair degree of accuracy (not necessarily extreme refinement), such problems as the best and most economical location, the most economical voltage, the most economical size of conductors, the most economical span between line supports, and the most economical phase correction or compensation, he is effecting no useful purpose in saying that the two above-mentioned statements (as regards the conductor) do not apply to him, or that one or the other is wide of the mark. The writer's experience forces him to admit of much truth in the above statements, and also to admit that, in general, they are fairly well borne out in actual practice. As in the case of a railroad or a water conduit, our mathematical minimum or averages are made subject to circumstances, local and otherwise, and our designs are made and the work is completed for conditions not in sight, and which from time to time call for changes in the original calculations, assumptions, probabilities, etc.

The most economical design of a transmission line is governed by many factors, the principal being load factor, transmission distance, cost of energy, and the annual expense of the investment. In general, big power capacity, high load factor, long transmission distance, high cost of energy, and low annual expense will result in a *high* most economical voltage ; and big power capacity, high load factor, short transmission distance, and low cost of energy will result in making the most economical size of conductors *large*. In the design of the most economical transmission line, the principal independent variables to be taken into account are : (a) distance of transmission ;

(*b*) amount of load ; (*c*) nature of load ; (*d*) cost of generating energy ; (*e*) number of lines ; (*f*) number of circuits ; (*g*) factor of safety ; (*h*) spacing of line supports ; (*i*) electrical system ; (*j*) transmission voltage ; (*k*) spacing of conductors, etc. ; (*l*) selling price of energy ; (*m*) interest and depreciation, etc.

The cost of a transmission line consists of (1) the generating costs of energy lost in transmission ; and (2) the allowances for interest and depreciation, etc., on the investment. By varying several independent variables, the cost will show a minimum for a *definite* power load (variable in practice) and *definite* transmission distance. The *real* problem is not purely a matter of solving for a theoretical minimum. The problem also demands practically an absolute security of service (best quality of the complete installation), as well as reasonably good line voltage regulation, etc. The former may mean (due to service requirements) a duplicate transmission line, a very high factor of safety, both electrical and mechanical, very high degree of protection against lightning and other disturbances, etc. The latter is always important, and is one of the requirements in the form of transmitted energy—its degree of variation will vary with the load, power factor, and transmission distance.

Because of the large capital outlay involved in the transmission of energy over long distances in great bulk at very high pressures, it is very important to locate, design, construct, and operate the line in the most economical manner. Industrial, financial, and other conditions will change the cost of the complete transmission system, and the value of energy, etc., and the constants assumed in the calculation for the determination of the most economical conditions (involving wayleaves, line supports, conductors, insulators, earth wires, land, buildings, transformers, etc.), may differ at one time to another sufficiently to show that the original calculations are widely different from the actual practical conditions.

The electrical efficiency of a power transmission line depends upon (1) ohmic loss in the line conductors ; (2) copper loss in the transformers at both ends of the line ; (3) magnetic loss in the transformers located at both ends of the line. The ohmic loss in the line is directly proportional to the square of the current and to the resistance of the conductors. In long and very high voltage lines the load current (*I*) may not always have an equal value along the line owing to the existence of capacity current (*I_c*). The ohmic loss is variable according to the change of load (power and power factor) and the temperature—the total power is dependent on the power factor. In the design of a transmission line a value is usually assumed for the amount of energy annually transmitted, which value takes into account a load factor something after the style, " the maximum load

is continuous for [*so many*] hours, the remaining [*so many*] hours being loaded about [*so much*] per cent. of the maximum." In actual practice the economical points upon which reasonings are based are rarely met, and where the most economical range of points has been adopted, the theoretical *true* minimum may be attained for but a very short period of the time, perhaps never ; if, for instance, certain economic problems were not fully considered or not properly understood in the light of future changes in the character and amount of load, etc. Usually the load, diversity, etc., are assumed long before they actually exist, and, in consequence, certain discrepancies arise in practically every design.

When the distance of transmission is increased, the cost due to the ohmic loss in the conductors will increase proportionally, but the cost due to the annual expense of line-supports, conductors, and insulators will not increase in the same proportion. The variation of load and power factor will have a great influence on the cost of transmission. When this latter factor is low, the efficiency will be proportionately low, and, as the total capacity is not being utilised efficiently, the annual expense will obviously increase in proportion. The cost of transmission is composed of an electrical cost (size, length, and spacing of conductors, the amount of load, the nature of the load, and the terminal apparatus, such as the efficiency of transformers, etc.), and a capital cost which represents the cost of line-supports, conductors, insulators, transformers, land, buildings, switchgear, etc., and the interest, depreciation, maintenance, taxes, etc. The interest on the investment will depend on both the value of the installation and the commercial conditions. The depreciation is usually carried forward from annual profits to provide for replacements. Maintenance, taken as a whole, will bear a definite proportion to the total amount of depreciation, hence the necessity of applying proper percentage values. The various items of depreciation and maintenance are not the same for the different parts of the complete undertaking. With regard to taxes, the usual taxes for a power transmission system are : (1) income tax ; (2) land tax ; (3) building tax. These taxes vary for different countries and sometimes for different localities in the same country. The cost due to annual expense will consist of several unequally durable and unequally costly items, the various items involved being way-leaves, line-supports, conductors, insulators, transformers, lightning arresters, switchgear, land, buildings, etc. It can therefore be seen that the importance of one item is estimated not only by the amount of money invested, but also by the coefficients of annual expense. Moreover, the cost of the complete system is dependent not only on the first cost, but also on the coefficients representing the interest,

depreciation, maintenance, etc., so that a low first cost for one part may very soon become a high cost item when the maintenance expenses become excessive on account of short durability, *i.e.*, poor quality of materials, etc., and inferior engineering and workmanship. Thus, in the long "chain" of variables, we see many reasons tending to prove the accuracy of the above statements and to show the great need and importance of preparing reliable figures of every item, based on a sound engineering experience, in order to obtain as nearly as possible the best and the most economical design.

The modern tower-line, carrying current at very high pressures, demands almost as careful study of its location as a railroad. In its proper location it demands a thorough knowledge of all the factors affecting the first cost, as well as subsequent operation; it requires a keen sense of observation, so that all the physical facts are accurately noted; that all advantages are taken and disadvantages avoided. In its location it is important to avoid, as far as possible, public highways, villages, telegraph and telephone lines, etc., at the same time keeping in mind such important points as the minimum distance of transmission, the minimum number of angles, the minimum altitude with minimum distance where corona is in evidence, also greatest protection from winds and lightning. Sometimes tower lines are cheaper if located across rolling country (hills and valleys) where longer spans and fewer towers can be used than in flat country. A careful study of all these minimum points of cost, etc., should decide the best and the most economical transmission line *location*.

In the location of a line it is preferable to first run a preliminary line (on paper), then project the location. When the located line is being staked out, a careful classification of the ground crossed should be made, showing rock outcrops, cultivated, bare, and/or swampy ground, etc. A line of fairly accurate levels should be run which will show breaks in ground of two or more feet. The field work should also show a record of all transverse slopes of more than about 10° . When the detailed profile has been completed, the line-support locations can be made in the office. One such office procedure for properly locating line-supports is to first make certain assumptions for loading—investigating sag and tension with a sudden drop in temperature or for a combination. A transmission line must be sufficiently high above the surface of the ground to conform with local and Government regulations, and the line-support must be built to resist (*a*) the weight of the structure itself; (*b*) vertical load due to the lines; (*c*) windage on its own structure; (*d*) transverse load applied at the top. These will determine the size of members in tension or in compression. of the support The vertical load

will be the total sum of the aggregate weight of conductors, insulators, sleet-coating, etc., and the downward pull of conductors due to the difference in the height of the points of support. The weight of a steel

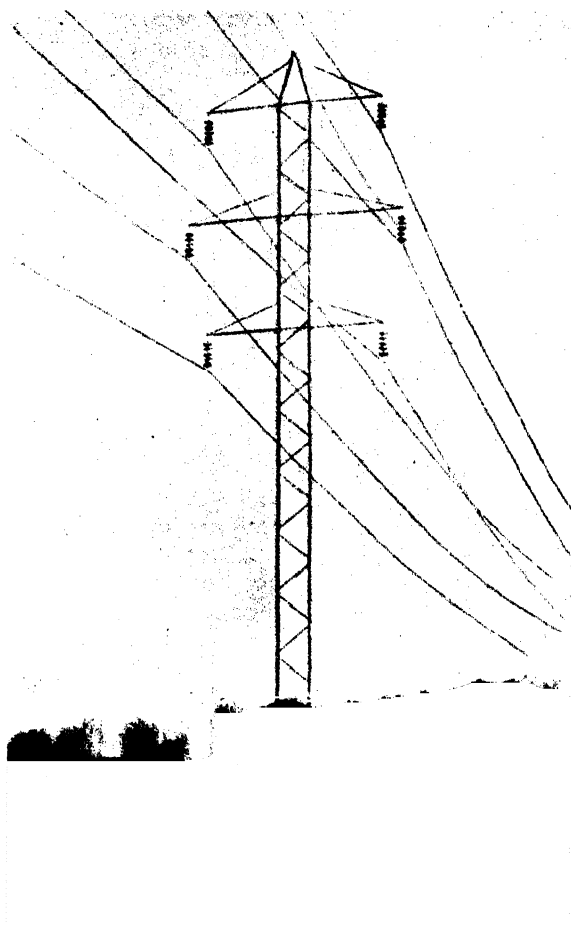


FIG. 30. — Showing very economical type of transmission line support.

(This steel laced channel line support is now used for voltages of 110,000 volts and above. The pole shown here is 75 ft. in height above the level of the ground, and is used with standard spaces of 600 ft. with anchor towers at every tenth pole.)

tower is very much dependent on the tension of the conductors either in the deflection of line or in the unbalance of tension. The cost of structural steel and line-support foundations will vary according to the commercial conditions. At the present time the type of steel

towers generally used is the *rigid* type. The weight of the line insulators is dependent on the voltage. For very high voltages the predominating requirement is insulating capacity, so that transmission voltage principally defines the cost of insulators, although the cost is mainly dependent on their weight. Duplicate lines should, preferably, be constructed upon separate supports and over a different route to avoid the chance of interruptions from storms, etc. Against this advantage is the increased cost of line-supports, wayleaves, patrolling, etc., but these may be offset by security of service. In the final staking out of line-supports in the field it is most important to engage a first-class man. An inexperienced man may destroy part or all of the good work accomplished in the office by greatly increasing the cost of line-supports (due to placing steel where it is not required), by endangering the line by mechanical weakness (due to placing ordinary supports instead of heavier supports at angles caused by changing the route to avoid obstacles not accounted for in the office location of the line), by causing an uplifting of the insulators sufficient to endanger or cause fracture of the cross-arms, by causing the clearance of conductors to extend below the allowable minimum, and, perhaps, finally causing a transmission line to be built that would, during the whole of its life, be an undue source of expense and danger.

For the electrical design itself, the *main* variables are voltage, power, and power factor; and, in view of the fact that a variation is dependent on the line constants and on the amount of and nature of load, it would seem that—considering all things—the engineer is reasonably justified in basing the design on the *average* values of these said variables. Certainly at times there occurs in actual practice a wide range of the different values of these variables. Nevertheless, we may take it that, in general, the design—taken purely from the point of view of energy losses in transmission and the annual expenses of the line investment—may take for the base data the averages of voltage, power, and power factor.

As the cost of transmission would, in certain cases (perhaps generally), vary but little for different span lengths, a determination of the most economical transmission line would be to:—

1. Decide on the most economical span for a definite voltage and definite size and kind of conductor.

When this has been done, the next move would be to:—

2. Determine the cost of the transmission line with the most economical span against the size of conductors for different values of transmission voltages. Then—

3. Plot these values to show one curve for each transmission voltage. Afterwards—

4. Mark off the minimum which determines *the most economical* size of conductors with *the most economical* span for a given voltage (standard voltages among others in order to show exactness). Then—

5. From the minimum points thus obtained for different voltages, draw a curve. Finally—

6. The curve thus drawn should show a minimum point for the most economical voltage of transmission, the most economical size of conductors, and the most economical spacing of line-supports

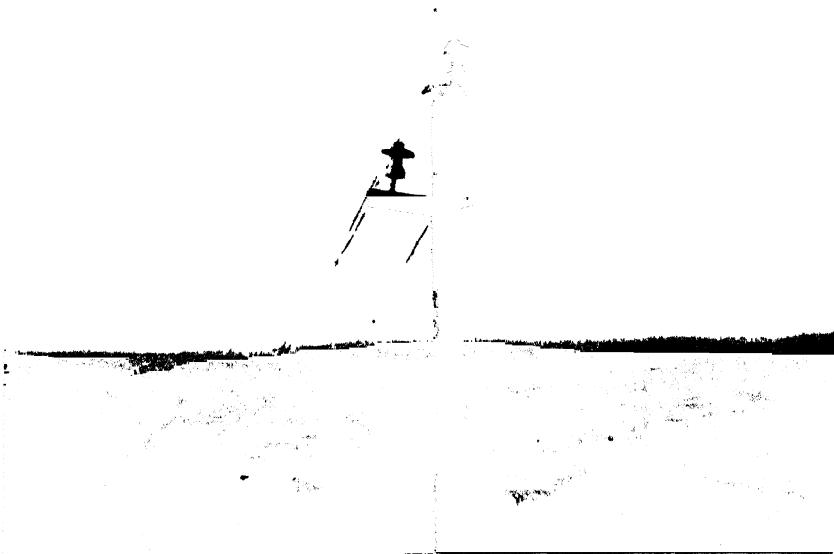


FIG. 31.—60,000-volt line trouble twenty years ago.

(This photograph was taken by the author in 1904 at the time he was engaged on the hydro-electric network of the (then) California Gas & Electric Corporation, California, U.S.A.)

In one way this picture illustrates the value of wood poles and X arms in a dry climate, as in this particular case no interruption of service occurred. Due to a leakage, the pin-hole burnt itself larger and larger until the X arm caught fire and finally burnt off, as shown.

In conclusion, although the final choice will in any case fall between two different commercial *standard* voltages and two different commercial *standard* sizes of conductors, both differing appreciably, one or the other of which must be selected for commercial and economic reasons, a point always to be borne in mind is that, in general, as in the case of the hydraulic waterways, an ample design is comparatively less expensive than a curtailed one; therefore use the larger of the two standard sizes.

When either voltage or size of conductors is increased, the cost of

transmitting energy will increase, due to the greater cost of the line, etc. The variation of the different parts of the cost with the change of voltage will be much more pronounced than that with the change of size of conductors. For a definite size of conductors the total cost of transmission will vary but little with a change of span, but it will vary greatly for a variation in the size of conductors ; also, for a definite span, the annual expense will increase with increased size of conductors. The cost of line-supports will increase with increased size of conductors, and the cost of insulators will increase with increased voltage.

Useful Information (Transmission Lines)

To determine the regulation :—

$$\text{Single-phase—} 2I(R \cos \phi) + (X \sin \phi)I = e'$$

$$\text{Three-phase—} \sqrt{3}I(R \cos \phi) + (X \sin \phi)I = e'.$$

Example (three-phase line) :—

E_r = voltage delivered	—	—	—	—	22,000 volts.
P_r = power delivered	—	—	—	—	6,000 kW.
$\cos \phi_r$ = power factor of the load	—	—	—	—	80 per cent.
l = transmission distance	—	—	—	—	10 miles.
s = conductor spacing	—	—	—	—	30 in.
f = frequency of system	—	—	—	—	50 cycles.
d = size of conductor (copper)	—	—	—	—	No. 5/0 S.W.G.

Then

$$I = (6,000 \times 1,000) \div (\sqrt{3} \times 22,000 \times 80) = 197 \text{ amperes ;}$$

$$R \text{ at } 25^\circ \text{ C.} = .31 \text{ ohm ; } X \text{ at } 50^\circ \text{ F.} = .526 \text{ ohm ;}$$

then voltage drop is

$$\sqrt{3} \times 197 (.31 \times .8) + (.526 \times .6) \times 10 = 1924 \text{ volts.}$$

$$\text{Regulation} = \frac{23924 - 22000}{22000} \times 100 = 9 \text{ per cent.}$$

Power loss is

$$p = \left(\frac{P}{E \cos \phi} \right)^2 R.$$

$$R \text{ per conductor (three-phase)} = 1/3 \times \frac{P}{(I)^2}.$$

$$\text{Power loss is } \frac{P \text{ per cent.}}{100} \times P \cos \phi$$

$$p \text{ per cent.} = (p/P) 100.$$

$$(\text{Three-phase}) = \frac{\sqrt{3}IR}{E \cos \phi}.$$

Voltage at generating end is

$$E_g = \sqrt{(E_r \cos \phi + IR)^2 + (E_r \sin \phi + IX)^2}.$$

Total current (including capacity current) is

$$I_t = \sqrt{(I \cos \phi)^2 + (I \sin \phi - I_c/2)^2}.$$

$$\cos \phi_r = I \cos \phi / I_t.$$

$$E_{\text{open-circuit}} = E_g + (I_t/2)X$$

CHAPTER XIII

POWER TRANSMISSION LINE CALCULATIONS

ABOUT the first question to be answered in every power transmission problem is that of relative cheapness of : (a) generating and transmitting energy to the point where it is needed ; or (b) generating energy at the point of demand. To satisfy the former case there are, at the present time, two available sources of power, namely : (1) the energy of falling water ; (2) the expansive force of steam. As a general rule the former condition is located by Nature and (of necessity and compulsion by Government) the best use is made of it ; the latter condition can as a general rule be located almost any place where there is ample condensing water.

In general it is found that a decision as to the system, frequency, voltage, size of conductor, supporting structures, etc., of power transmissions is dependent in some way or other on electric power systems already operating in the area or within transmission distance ; and that, as regards the distance of transmission, economy only is the dictator and not the distance.

At the present time two distinct types of overhead transmission line construction are being developed, one having line-supports which are rigid in all directions, and the other having line supports which are rigid and semi-flexible ; that is to say (as regards the latter type), lines where there are rigid supports as well as intermediate supports which are semi-flexible lengthwise of the line. The rigid steel tower transmission line construction is at the present time mostly confined to big power lines operating at very high voltage. All such overhead lines are of the best grade of construction.

In order to determine the best and most economical overhead transmission line supporting structures it is necessary to determine either the *total* first cost of construction for the entire line, or to bring the total cost down to a value per unit length of line. It is not only necessary to include the cost of all line materials, insulators, fittings, etc., all of which will vary with the number of line-supports (*i.e.*, with the spans, angles, etc.), but it is also necessary to compare the several costs of different types of line-supports.

In the selection of line-supports (poles and frames) we may have to decide upon any one of the following conditions :—

1. To obtain from manufacturers or dealers the necessary specifications, quotations, etc., of *tested* and/or *approved* poles, towers, or frames of standard designs. The selected line-supports would then be spaced in the line so that they would use most of their available strength. Or,

2. The company may consider it better or more economical to design their own line-supports. In this case they would first decide on the distance between the line-supports (poles or frames) for the proper size of conductors, etc., and then design line-supports which would theoretically have available not less than the required strength for straight runs and others for corners or angles in the line.

As regards the former condition, this is usually handled in a satisfactory manner by specialists, just as in the case of most other engineering matters. The latter condition is usually not so well favoured with first-class knowledge of materials, arrangement, design, and construction, etc., and it is doubtful whether circumstances would usually make it worth while experimenting with untried and/or untested designs. In any case, the engineer concerned must be quite familiar with all the existing safety laws and customs; and, moreover, he must be able to: (a) determine the total conductor loads; (b) determine the total structure loads; (c) estimate (as required) the wind or/and ice and sleet loads to a fair degree of accuracy; (d) select suitable designs; (e) select suitable materials, etc.; and (f) apply a proper safety factor to each of his designs.

The heaviest load on any given pole line is that caused by the longitudinal stress of all the conductors, which clearly shows that the size of conductor limits the span-length. These stresses are usually taken care of by suitable dead-end structures or by suitable guys placed at appropriate intervals along the line. With increasing temperature the sag increases, producing a reduction of this stress.

With fairly long spans (long spans for wooden poles) few wood (single) poles have sufficient strength to withstand the longitudinal stress without the help of stay-wires or cables. For such spans the best grade of wood pole in its best condition will not ordinarily sustain the required percentage of the actual longitudinal stress unless it is stayed. There is also the question of rot to be accounted for, which penetrates a wood pole at a point just below the ground line (to about 1 ft. below). The rate of decay depends upon the kind of wood, treatment of the wood, climate, changes in moisture, and upon the kind of soil. Certain treatments are a pretence only, and very often the rate of rotting of a treated pole, when once it starts, is about as rapid as though the pole had not been treated. Treated wood poles stand very favourable in this country; a rather long life is about thirty years, whereas in North America their average life is nearer fifteen years. Perhaps the

greatest objection to wood poles is the maximum permissible span-lengths allowable as compared with steel.

Earthed wires are sometimes placed upon wood pole lines to prevent direct strokes of lightning from shattering the poles. The addition of such earthed wires or cables will increase the transversal pole load, requiring either a closer pole-spacing or a heavier and more expensive line-support. The use of an overhead earthed wire will assist longitudinal rigidity (see p. 215).

In the event of a broken line-conductor or conductors, and to permit the unbalanced stress to distribute and equalise itself over a number of spans and so immediately reduce the overloaded condition of any one pole or tower, it is usually advisable to have the intermediate line-supports as flexible as practicable, *i.e.*, flexibility in the direction of the line. Each of the semi-flexible structures will move slightly at the top until equilibrium has been established according to the new conditions set up. Unless the line is self-supporting when a certain number of conductors are broken, certain of the supports should be (guyed) stayed lengthwise of the line. Where a guy or stay is used * on a line-support which does not have surplus transversal strength, stay-wires should be arranged cross-wise of the line. If a larger sized support is not used at such corners or angles in a line then the adjacent spans should be shortened. For all angles greater than about 14° , a larger size pole or tower should be used than those for a straight run; where this is not possible, the total dead-end stress should be taken up by the available structures, struts, and/or stays. It is very desirable to have a straight route, or, failing this, to avoid abrupt angles in the line.

The choice of a factor of safety requires a good general knowledge of the materials used, climatic and other conditions in the locality; and it involves many uncertainties such as: (a) uncertainty of the amount and the effects of loads and distribution of the resulting stresses; (b) the ultimate weakening of any of the structures due to corrosion; (c) defects in the materials and poor workmanship; (d) probable life of each of the materials used; (e) character of the service required at the present time; (f) character of the service which will probably be required in the future; (g) possible future changes in the power capacity of the line, etc. The factor of safety should allow for errors in designing, corrosion, defective materials, and workmanship, the assumed loads and the improbable loads. The factors of safety are different for the different countries on account of the great variation in the conditions producing the actual loads, etc. For instance, in Africa no ice coating is allowed for.

* A factor of safety no less than three should be used.

The annual cost of any given transmission line depends on the following : (1) maintenance cost ; (2) operating cost ; (3) depreciation and taxes ; (4) reserve for contingencies ; and (5) interest on first cost. We therefore see, in comparing several types and/or combinations of construction giving the most economical arrangement and materials from well-tried types and combinations, that, in so far as the line-supports are concerned, the most economical line-supports should have the lowest annual cost ; this would of course also refer to the complete line. That is to say, it would ordinarily include : (*a*) all expense incurred for repairs and replacements ; (*b*) the cost of energy absorbed in the line, cost of inspection and patrolling, etc.

Conductor stresses are, in no small way, dependent upon climatic conditions. The maximum loading to be known in making a determination of the sags corresponding to a given limiting longitudinal stress in the conductors must be heavier when subjected to ice and sleet loads than otherwise ; that is to say, the maximum loadings of line-conductors operating in Canada would be greater than for lines operating in Africa. Therefore, in computing the loads upon line-supports, it is necessary to know where the poles or towers are going, and then to determine the maximum loadings for the particular climatic conditions. Vertical loadings upon the line-supports and cross-arms should be taken as equal to their own weight, plus the weight of the ice or sleet-covered conductors.

Before the size and spacing of line-supports (poles, " A " and " H " frames,* and towers) can be determined, the size of the line-conductors must first be determined or assumed. The span-length is limited by the size of conductors, and the size of conductors is controlled by the voltage drop, the power loss, the carrying capacity, or their cost. In the design of an overhead power transmission line, the size and spacing of conductors usually are determined for the reason that the working voltage, voltage regulation, and other factors place a limit on both. The *exact* size of conductor is not dependent on the theoretical area as determined by accurate formulæ, but upon the choice of one of the nearest two consecutive commercial sizes of conductor. This, no doubt, is one of the reasons why the size of conductor is assumed, because it is from the commercial size and not the theoretically determined size that the line losses, etc., are computed. Thus the choice of a line-conductor is limited to such standard sizes as are manufactured commercially. Also, a choice

* These two types of line-supports are widely used in this country ; depending on the line voltage, spans up to 500 ft. may be used for ordinary three-phase, single-circuit lines.

of transmission voltage (upon which the size of conductor for a given power depends) is limited in the same way; the higher the line-voltage for a given power the smaller will be the size of conductor.

It seldom happens that when designing a transmission line the exact amount of power required to be transmitted and the load power factor and certain other conditions are accurately known. Moreover, the size of conductor is determined from one (and only one) set of conditions, such as the averages of power, voltage, and power factor, etc., and the results obtained by the use of certain charts (such as the one presented here) are well within the limits of accuracy usually demanded for practical work. Neglecting the effect of corona, the proper size of conductor depends upon the following main conditions: (1) voltage at the load; (2) the power delivered; (3) the permissible voltage drop or the power loss, whichever is the controlling factor; (4) the distance of transmission; and (5) the power factor of the load. The permissible voltage drop is dependent upon the frequency of the system and the separation of the line-conductors. In certain of the Colonies very high voltages are employed, so that the effect of electrostatic capacity (charging current) will be appreciable for transmission lines above 70 miles in length and receiving voltages higher than 70,000 volts at 50 cycles per second. In such cases the total power loss in the line, as based upon the theory of transmission lines, becomes slightly different to the load (current) power loss.

For reasons of mechanical strength of the conductors it is not advisable to use a copper conductor smaller than 0.160 in. diameter, or 0.276 in. diameter for *long* spans; but for relatively short spans (usually for wood pole lines) the smallest size of conductor that may be used in regions not subject to ice and sleet is nearer 0.140 in. diameter. As the line-inductance increases very rapidly with the size of conductor and the frequency (as will be seen by the chart), it may be advisable to divide the line into two or more circuits. Two or more of such parallel circuits will have a much smaller inductive-volts drop, will give lower operating costs, and will provide greater security from interruptions. The largest size of copper conductor *commonly* used is around 0.400 in. diameter for a 50-cycle system and 0.500 in. diameter for a 25-cycle system.

The size of conductor is mainly controlled by the voltage drop. The voltage drop (including lines operating up to the highest voltage in this country at the present time) depends upon the resistance-volts drop (IR) and the inductive-volts drop (IX) and upon the power-

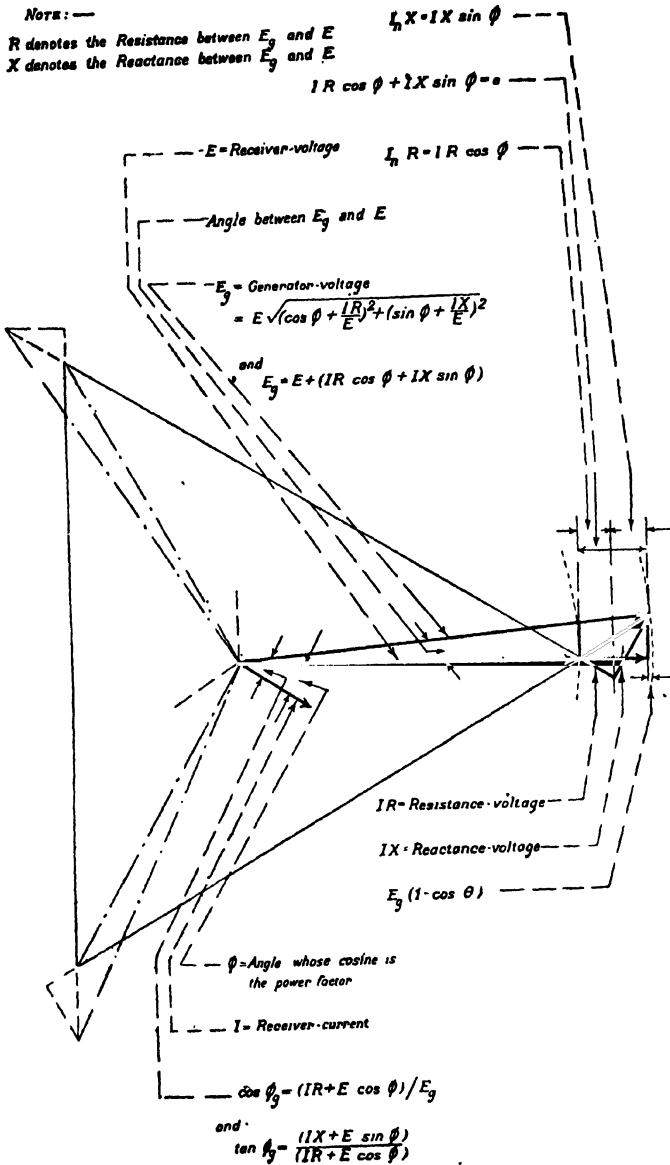


FIG. 32.—Diagram for determining voltage regulation of three-phase transmission lines.

factor of the load ($\cos \phi$); thus for all such lines the total voltage drop can be expressed :—

$$e = E_g - E_r; \text{ or } e = (IR \times \cos \phi) + (IX \times \sin \phi)I,$$

where E_g = voltage at the generating or sending end.

E_r = voltage at the receiving end of the line.

l = transmission distance in miles.

I = current of load in amperes.

R = ohmic resistance per mile of conductor.

X = inductive reactance per mile of conductor.

$\cos \phi$ = power factor of the load.

This voltage drop is usually expressed as a percentage, *i.e.*, a percentage of E_r .

The distance of transmission, the delivered power and voltage, and the load power factor form the principal controlling factors in the determination of line-regulation, power loss, size of conductor, etc. They are the factors invariably assumed as being fixed in value. In computing the size of conductor, or, what is better, in computing the voltage drop or the voltage-regulation for any given size of conductor, the above-mentioned controlling factors are first assumed, as is also the frequency, the kind of system, and the spacing of the line-conductors. As regards the transmission voltage this is governed by many factors such as : (1) voltage of existing neighbouring systems; (2) amount of power to be delivered; (3) first cost of necessary terminal apparatus for different voltages; (4) relative cost of power generated and transmitted; (5) probable load factor; (6) probable cost of replacements; (7) desirability of operating duplicate lines; (8) annual cost of transmission; and (9) the transmission distance. The higher the voltage the greater will be the cost of the terminal apparatus as well as the line construction and equipment. On the other hand, the size of conductor will be smaller for a given amount of power. As regards the most economical design of a transmission line this is governed by such principal factors as the kVA. capacity, load factor, transmission distance, cost of energy, and the annual expense of the investment.

Sometimes the transmission voltage or the receiving voltage is computed from approximate formulæ. The most accurate formulæ for voltage of transmission includes the total capacity in kVA., the length of transmission in miles, and the cost of energy per kW.-year. The transmission voltage should increase when the kVA. capacity increases, similarly as when the transmission distance is increased, and also as the cost of energy increases. In general, the power to be delivered is the most important factor in the choice of line-voltage. The old rule for three-phase lines was to take 1,000 volts per mile

irrespective of the amount of power to be delivered. An improvement on this rule for *line-voltage* is the following :—

TABLE XVIII.—SHOWING COEFFICIENT (k) FOR DIFFERENT LENGTHS OF LINE IN MILES

Distance in Miles (L).	Multiplier per 1,000 Volts (k).	Distance in Miles (L).	Multiplier per 1,000 Volts (k).
30	1.034	40	.9750
20	1.125	50	.9275
15	1.178	60	.9000
10	1.278	70	.8750
6.6	1.400	80	.8500

That is to say (neglecting for the moment the effect of line kVA. capacity), we have for a 30-mile line a volts-mile value of 1.034; for a 15-mile line, 1.178; and for a 60-mile line, .909 = 909 volts per mile of line. However, apart from the question of volts-miles, a very important question in the choice of line-voltage is the amount of power to be transmitted over the line. With this factor included, we may, as a *first trial* determination of line-voltage (not the most economical voltage) use the following empirical formula * :—

$$E = k + (.00001 P_e) / \text{kilovolts},$$

where k = multiplier per 1,000 volts-miles; P_e = kVA. transmitted over the line; l = distance of transmission in miles.

Thus, for a 40-mile three-phase overhead transmission line supplying 3,750 kVA., we have

$$E = (.975 + .00001 \times 3,750) \times 40 = 40.5 \text{ kV.}$$

The receiving voltage E_r in this case should be 40,000 volts = E_r .

For a three-phase line 50 miles long delivering 1,600 kW. at 80 per cent. power factor, we have

$$E = .9275 + (.00001 \times [1,600/.8]) \times 50 = 47.37 \text{ kV.}$$

This voltage is high, and a standard receiving voltage E_r no higher than 40,000 volts should be used.

If it is required to deliver the same load (2,000 kVA.) a distance of 20 miles (three-phase) we get

$$E = (1.125 + .00001 \times 2,000) \times 20 = 22.9 \text{ kV.}$$

The nearest standard receiving voltage is 20,000 volts = E_r .

* Cost of electric energy per kVA. year is omitted from this formula. For fixed values of P_e and l , an increase in the cost of energy per kVA. year will result in an increase in the most economical voltage E , the size of conductor, but a decrease in the cost of transmission, which represents the cost of energy at the generating end.

Generally, large kVA. capacity, high load factor, long transmission distance, high cost of energy, and small annual expense of investment will make the most economical line-voltage *high*.

Standard *receiving* voltages are often used with the following mentioned transmission distances:—

TABLE XIX.—SHOWING THE APPROXIMATE RANGE IN VOLTAGES (E_r) FOR DIFFERENT LENGTHS OF LINE IN MILES

Distance (l) in Miles.	Standard Voltage ($kV' = E_r$).	Distance (l) in Miles.	Standard Voltage ($kV' = E_r$).
30 to 50	40 to 60	10 to 15	15 to 20
20 „ 30	30 „ 40	5 „ 10	6 „ 10
15 „ 10	20 „ 30	3 „ 5	3 „ 6

In a general way we may take it that the most economical line-voltage for any length of line is that voltage which gives the lowest first cost when conductors, insulators, supporting structures, terminal apparatus, erection costs and maintenance are properly considered. In computing the line-voltage, it is desirable to estimate the cost of line and terminal apparatus at two or three nearest standard voltages; these will show whether line insulators, etc., or terminal apparatus, etc., will indicate the choice of line-voltage. The chosen size of conductor should show the lowest annual charges for a particular line-voltage; if the voltage is raised, the annual charges and first cost of conductors will both decrease. The line may be long enough to prevent the cost of transformers, switches, lightning arresters, etc., at the terminal ends controlling the choice of line-voltage. Where lines are short the cost of terminal apparatus is usually the limiting factor instead of the line itself. The cost of the supporting structures may also increase to such a point where they outweigh any saving in conductor cost which might be obtained by a further increase in line-voltage. For very high voltage and long lines, while the line-current is decreasing with an increasing voltage, the charging current is increasing.

The amount of copper in any transmission line varies inversely as the square of the voltage, *i.e.*, as E^2 . Thus, if the voltage is doubled, the weight of the line-conductors will be reduced to one-fourth (neglecting the effect of charging current, leakage, and corona). Also, if the line-voltage is doubled the current will be halved for a given delivered load, therefore, for the same spacing of line-conductors the reactance-volts drop will be halved, resulting in one-fourth the percentage of the reactance-volts drop. For the same

amount of power transmitted at the same efficiency and the same voltage the weight of copper will increase directly as the square of the distance.

We may now consider the selection of line-supports for various spans ; that is to say, we can now determine the size and lengths of various line-supports (poles,* towers, " H " or " A " frames). To do so we require to know : (1) total over-all height of line support ; in this matter it is of interest to note that maximum length may or may not be determined by the sag rather than by the strength of the support (pole or tower) ; (2) length of support required ; (3) maximum clearance above the surface of the ground ; (4) sag of the conductors at a given temperature ; (5) separation of the line-conductors ; (6) total transverse conductor load, *i.e.*, effective safe transversal load strength of support ; (7) size of support required ; (8) *net* safe transverse load strength. In order to determine the correct size of stay-wire or cable, it is first necessary to know the total dead-end stress of the conductors. The most economical method of providing for extra strength at angles in the line is to use stay-wires. Where these are used, we may require to know : (a) size of side stay-wire or cable at an angle in the line ; (b) size of stay designed so as to carry the stress caused by one or more broken conductors ; (c) size of stays for anchoring the line. Now, assuming that the size of conductors has been chosen and that the span-lengths are known, then we may start with the design. That is to say, we may, depending on the type of line-structure (steel or wood) :—

1. Obtain the moment of resistance by multiplying the safe load in lbs. by the height in feet of point of application of load above the surface of the ground.

This may be at one or more points. A line-support must be sufficiently high above the ground where the voltage is high and the sag of the line-conductors is great on account of long span between the line-supports. The effective height is the height of the point where the resultant force will apply ; it consists of three parts, one of which is proportional to the line-voltage, another to the square of the span-length, and the third to the limit of height imposed by law, etc., which is independent of line-voltage, span-length, spacing of conductors, and sag.

* The economic span for wood poles lies between 200 to 300 ft. ; for steel towers with narrow base, 400 to 500 ft. ; and for wide base steel towers, 600 to 800 ft. ; for " A " or " H " frames or poles, 200 up to 450 ft. The present tendency is toward the use of longer spans than has been the custom in the past. The *most economical span* for the wide base steel tower (three-phase) line carrying 10,000 kVA. 50 miles, at from 60 to 100 kV., is around 600 ft. About the best means for decreasing *wayleave* difficulties is to increase the span. As sleet will not form with heavy winds, the longer spans have the advantage of relatively less loading due to ice-coating of conductors.

2. Obtain the moment load in lbs., *i.e.*, the moment in feet-lbs.* due to the load. This can be obtained from the expression :

$$s(W'n'h' + W''n''h'' + \dots),$$

where s = span between line-supports in feet.

W = horizontal wind load (in lbs.) on a conductor per foot of length.

n' , n'' = number of line-conductors of the same weight on the same cross-arm.

h' , h'' = height in feet of point of application of load above the surface of the ground.

A line-support has to resist, in general, four different kinds of mechanical loads, namely : (a) weight of the complete line-support ; (b) windage on the line-support ; (c) vertical load due to the conductors, etc. ; (d) transverse load applied at the top of the line-support by the windage on the conductors and the unbalanced pull of the conductors.

3. Obtain the working stress in a conductor. This can be obtained from the expression :

$$(ws^2) \div (8g),$$

where w = total load (in lbs.) on a conductor per foot of length.

s = span between line-supports in feet.

g = sag in feet of a conductor.

4. Obtain the sag (in feet) of a conductor. This can be obtained from the expression :

$$(ws^2) \div (8C),$$

where w = total load (in lbs.) on a conductor per foot of length.

s = span between line-supports in feet.

C = working stress (in lbs.) of a conductor.

5. Obtain the total dead-end load due to the conductors in lbs. This may be obtained from the expression :

$$Cn + C'n' + C''n'' + \dots,$$

where C = working stress (in lbs.) of a conductor, *i.e.*, tension at the lowest point of the conductor in lbs.

n = number of conductors of the same weight on the same cross-arm.

6. Obtain the transverse load on a line-support due to an angle in the line. This may be obtained from the expression :

$$2w'' \sin (\phi/2),$$

where w'' = total dead-end load due to conductors in lbs.

ϕ = horizontal deflection or angle in the line.

* B.O.T. regulations up to recently assumed the pressure acting in the horizontal direction with a force of 25 lbs. per sq. ft. of conductor. At present it is 8.0 lbs. per sq. ft. *plus* $\frac{1}{2}$ in. ice on the conductors at 22° F.

The transverse load which is applied at the top of a line-support by the conductors consists of two components, one in the direction of the line due to the tension of the conductors, the other in the direction vertical to the above-mentioned, and is due to the windage and the side pull of conductors. The weight or tonnage of a steel line-support is very much dependent at all times on the tension of conductors either in the unbalance of tension or in the deflection of the line.

7. Obtain the vertical stress on a line-support due to a stay-wire. This can be obtained from the expression :

$$s' \cos \phi',$$

where s' = stress (in lbs.) in a stay.

ϕ' = vertical angle between stay-wire and the line-support.

8. Obtain the stress (in lbs.) in stay-wire or cable at a dead-end. This can be obtained from the expression :

$$w'' \div \sin \phi',$$

where w'' = total dead-end load due to conductor in lbs.

ϕ' = vertical angle between stay-wire and the line-support.

"H" frames and other forms of dead-end frames or towers may be used in the place of stay-wires, but this method of anchorage is always more expensive.

9. Obtain the stress (in lbs.) in a stay-wire or cable at an angle. This can be obtained from the expression :

$$w' \div \sin \phi',$$

where w' = resultant load (in lbs.) due to any number of concurrent forces.

ϕ' = vertical angle between stay-wire and the line-support.

The most economical and practical method of anchoring the conductors and taking care of the extra strength at angles is to use stay-wires. Where line-supports are stayed at corners or angles of a line they should be of larger size. If they are of the same size as the line-supports used in the remainder of the line, each of the adjacent spans should be shortened by a certain percentage (depending on the angle), so as to give the stayed line-support sufficient strength to carry the additional stress produced by the stay-wire or wires, whichever is used. There is also the question of liberal tension or tightening of the stay-cable at these points, which may cause unnecessary load upon the line-support. It is considered good practice to anchor straight runs at about every ten pole lengths and provide suitable pole structures at these "dead-ending" points in the line. Straight runs are generally the safest, cheapest, and best, both in first cost and operating expenses, hence every endeavour should be made to secure wayleaves for straight runs on reasonable terms.

TABLE XX.—PHYSICAL PROPERTIES OF WIRES AND CABLES

Conductor.	Lbs. per Square Inch.				Standard Conductivity.	Weight in Lbs. per 0.001 Sq. In. per Lineal Foot.
	Solid Wire.		Stranded Cable.			
	Ultimate Strength.	Elastic Limit.	Ultimate Strength.	Elastic Limit.		
Hard drawn copper -	{ 43,000 to 66,000	{ 34,400 to 51,500	{ 47,000 to 60,000	{ 37,000 to 48,000	96 to 99	.00389
Steel-aluminium wire -	160,000	130,000	...	...	8.7	.0034
Aluminium cable -	...	...	24,000	14,000	61 to 63	.00116

TABLE XXA

	Lbs. per Square Inch.				Specific Gravity.
	Solid Wire.		Stranded Cable.		
	Maximum Working Stress.	Usual Working Stress.	Maximum Working Stress.	Usual Working Stress.	
Hard drawn copper	22,000 to 28,000	20,000 to 26,000	19,000 to 27,000	18,000 to 23,000	8.96
Steel-aluminium wire	80,000	117,000	...	...	7.85
Aluminium cable	...	...	12,000	12,600	2.68

TABLE XXB

	Copper.	Aluminium.
Ratio of weight for equal area - - -	3.3	1.0
Ratio of diameter for equal resistance - -	1.0	1.29
Ratio of area for equal resistance - - -	1.0	1.66
Weight per mile (1 sq. in. cross-section) in lbs. -	20,590	6,195

If the amount of power is large, the frequency high, the transmission line long, and the power factor of the load low, the inductive voltage will be high. For a given length of line and size of conductors an increase in the frequency and/or the spacing of line-conductors will result in an increase in the inductive voltage. On the other hand, an increase in the size of conductors for a definite

frequency, length of line, and spacing of conductors, will result in a decrease in the line-inductance voltage but an increase of charging-current in the line. When the demand for power is large at high load factor, the advantage of using phase-compensators will usually be quite beneficial. On the other hand, when the load is quite variable and the load factor low, phase-compensation at light load will usually result in but little saving of power loss in the line. When the load factor is high, the ohmic loss will tend to increase and the annual expense *decrease*, resulting in a high voltage and large size of conductors for the line. On the other hand, when the power factor is low, the efficiency is proportionally low, and the annual expense will *increase* in proportion. It is quite clear that phase-compensation is economical when the efficiency of transmission is improved and the annual expense of the transmission (per unit of energy) is decreased thereby, but not otherwise.

Capacitance determines the charging-current in a line. It depends on spacing, diameter, and arrangement of the line-conductors, frequency, and length of the line. The two serious conditions to be met with in practice are :—

1. The charging-current flowing into a long line must be supplied by the generators and may be equal to or greater than the energy current when the line is fully loaded.

2. When full voltage is impressed on a transmission line at the generating end, and the receiving end of the line is left open, the voltage at the receiving end of a very long line may rise to such a value as to cause serious damage.

In other words, on open circuit there may be a dangerous rise in voltage on the line near and at the receiving end and a dangerous flow of current at the generating end. The effect of charging-current on voltage-regulation will be considerably greater at 50 cycles than at 25 cycles. Its effect upon the voltage-regulation will increase with increased length of transmission. Assuming that an error of one-half of 1 per cent. is allowable, then the accompanying chart is sufficiently accurate for 50-cycle power lines up to about 50 miles in length and line voltages up to 60,000 volts.

In the design of a transmission system, the voltage-regulation must be within such limits on all parts of the system that satisfactory service is secured, and, at the same time, all the power transformers obtain proper exciting voltages. Service for lighting is very exacting, since a 2 per cent. variation in voltage causes a change of approximately 8 per cent. in candle power. Since the power consumed by any load falls off approximately as E^2 , it is important that the voltage be maintained as high as is consistent with satisfactory service. The consumer is concerned with voltage-regulation ; the electric supply

company is concerned with the loss of energy ; * and the line itself with its own integrity.

The percentage voltage-regulation depends upon the separation of the line-conductors, the arrangement of the conductors, the frequency, the load power factor, the ohmic resistance, and the length of line, etc. The length of line referred to here is the excess length of conductor over the transmission distance or span-length. The percentage regulation is the voltage drop expressed as a percentage of the receiving volts $[(\sqrt{3}e/E_r)100]$. The area of the conductor required varies inversely as the percentage regulation or percentage energy loss (whichever is taken as the controlling factor), directly with the length of conductor (not length of route), directly with the load, and inversely as the square of the voltage E^2 . For hydro-electric systems, usually percentage voltage-regulation is the controlling factor and not percentage energy loss. Apart from the question of voltage or spacing, the I^2R loss will be the loss occurring in each conductor, and for a balanced three-phase system, the total losses in the line will be

$$(3I)^2 \times \frac{R}{3} \text{ or } 3I^2R,$$

or, for equal total line losses, $3I^2R = 2(\sqrt{3}I)^2R'$; wherein R' is resistance of each conductor on the single-phase line, and R is the resistance of each conductor on the three-phase line.

The size of conductor is, in general, controlled by the allowable voltage drop in the line. In systems employing phase-compensators (synchronous condensers, etc.) the choice of size of conductor will be controlled rather by the power loss than by the inherent voltage-regulation; *i.e.*, the difference of voltage between the generating terminals and the load terminals. As phase-compensators are not generally used, it may be said that usually one of the principal questions concerning a transmission line is, by what percentage must the generated voltage exceed the receiver voltage, not only for unity power factor but for loads of any power factor? As compared with all other line calculations, the two quantities most often computed in the design for operation of transmission lines are the voltage drop and the power loss. The latter is important in determining the economy of a design. In very high voltage lines charging-current also has an effect, but as such problems will be of rare occurrence in this country, further mention is considered unnecessary. It may be noted that, generally, large kVA. capacity, high load factor, short transmission line, and high cost of energy will make the most economical size of conductor large.

* In the determination of the size of conductor for power transmission lines loss of energy in the line and/or voltage-regulation are usually assumed to range between 7 and 12 per cent.

The accompanying chart and Table XXII. include the maximum and minimum sizes of copper conductors recommended for overhead power transmission work, *i.e.*, for conditions ranging from a *minimum* of about

$d = .176$ in., $A = .02433$ sq. in. (solid) for $S = 18$ in. (see p. 251),

$d = .204$ in., $A = .025$ sq. in. (stranded) for $S = 18$ in. („),

to a *maximum* of about

$d = .500$ in., $A = .1963$ sq. in. (solid) for $S = 144$ in. (see p. 251),

$d = .574$ in., $A = .20$ sq. in. (stranded) for $S = 144$ in. („),

and for ratios of S/d ranging from 25 to 1,000.

Wherein :

d = diameter of conductor in inch.

A = area of conductor in square inch.

S = separation of conductors in inches.

In fact, the chart can be used for overhead transmission and distribution lines as well as for insulated (underground) cable work in general. As regards the latter, take the following :—

Example.—In a certain part of a town distribution it is required to run $\frac{1}{2}$ mile of single-core copper cable of .025 sq. in. cross-section (.204 in. diameter); the system is 50 cycles, single-phase, and the cables are spaced 5 in. between centres. Find the reactance-volts per ampere per mile of cable. In this case, $S/d = 24.5$. From the chart, follow value $S/d = 5.0/0.204 = 24.5$ vertically to the intersection of the radial 50-cycle line (*upper line* in every case for stranded conductors); we find the value 0.43 volt per ampere per mile, hence, for $\frac{1}{2}$ mile of cable we have 0.215 volt per ampere per mile. Now, take an underground insulated copper wire of 0.176 in. diameter (No 7 S.W.G.) spaced 3.5 in. between centres; length of wire and frequency remaining the same as above-mentioned. Then, $S/d = 3.5/0.176 = 20$. From the chart, resistance-volts drop is seen to be 0.398 volt per ampere per mile of single conductor, or 0.199 volt for the length of wire given.

Conductor-spacings are dependent upon conductor arrangement, voltages, and sags, *i.e.*, *spans*. The sag of each conductor in turn depends upon the size, strength, and weight of the conductor, the ice, wind, and sleet loads, and the span-length. The deflection of aluminium conductor necessitates a larger spacing of the conductors; a greater sag is required than for copper or steel-aluminium, which may also demand a higher line-support (probably 15 per cent. higher). A shorter span is required for the aluminium conductor, and short spans are usually more affected by high winds than long spans are. The larger spacing of the *all*-aluminium conductor does not apply to steel-aluminium conductors which can be given much less sag and

longer spans because of its relatively greater strength. Conductors supported on pin-type insulators require less spacing than conductors strung on suspension insulators; the minimum separation of the latter method of stringing should be increased about 25 per cent. above the values given in the following table, *i.e.*, Table XXI. Very often conductors on long spans swing in synchronism and very often conductors on short spans swing out of synchronism. As wind will deflect copper conductors about 30 per cent. less from the vertical position to that of steel-aluminium conductors, the latter should have a greater separation for the same sag. Copper conductors larger than 0.500 in. diameter, strung on pin-type insulators, should be given a less spacing (for equal voltages) than smaller sizes; this rule applying, with relative modification, to other metals. The sine of the angle of deflection of copper conductors ranges from about 0.90 for a 0.192 in. diameter to about 0.72 for a 0.400 in. diameter conductor. For all the ordinarily used sizes of copper and aluminium conductors the ratio of wind loading to the weight of conductor is nearly in the ratio of 4 to 1 in favour of copper.

Where conductors are more than about 1 in. apart between centres, the inductive-reactance cannot be neglected and the common *direct-current* formula, $A = (Ik\ell)/e$, cannot be used without certain appreciable error; this formula gives the size A of conductor to carry a given current I a given distance ℓ with an assumed voltage drop e , the value of k is a *constant*. The inductance for any spacing S of conductors and frequency f can be found from the chart by dividing the reactance values by $6.283 f$. For a given size of conductor the reactance is varied by changing the spacing of the conductors, which changes the ratio S/d ; changing only the size of conductor will have but little effect upon the reactance. We may express the conductor-spacing (using copper and pin-type insulators) in terms of: (a) 1 in. per 1,000 volts; or (b) 1 ft. per 10,000 volts; with a minimum spacing in each case of from 15 to 30 in., which would depend on the span, size, and arrangement of conductors, metal used, climate and locality, voltage and sag. Approximate formulæ for *minimum* spacing of vertical spacing of wires in inches are:—

$$\begin{aligned} S &= 10 + (0.8 E_v/1,000) && - && - && \text{copper.} \\ &= 10 + (E_v/1,000) && - && - && \text{alu.-steel.} \end{aligned}$$

However, these are but rough rules, and the following empirical formula is suggested for *minimum* spacing (delta arrangement with base of triangle vertical) of three-phase (3-wire) circuits and (vertical formation) of single-phase (2-wire) circuits:—

$$S = 0.001 EK = (kV) K \text{ in.,}$$

where E = line voltage; kV = kilovolts; S = spacing of conductors

in inches (*with base of triangle vertical*); K = coefficient. The values of K for different voltages and sags (using the pin-type insulator) are :—

TABLE XXI.—VALUES OF COEFFICIENT (K) FOR DIFFERENT SAGS IN FEET

(E) Line Voltage.	Sag of Conductor in Feet.									
	2 to 4		6		8		10		12	
	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)	(a)	(b)
3,000 to 6,000	2.3	2.8	3.0	3.6	3.3	4.3	4.5	5.0	5.0	5.8
10,000	1.6	2.0	2.0	2.4	2.4	2.9	2.8	3.4	3.2	3.8
11,000	1.54	1.9	1.8	2.27	2.18	2.72	2.54	3.18	2.9	3.45
15,000	1.33	1.6	1.6	1.86	1.70	2.13	1.95	2.46	2.26	2.66
20,000	1.15	1.4	1.25	1.55	1.45	1.75	1.65	1.95	1.85	2.15
22,000	1.09	1.36	1.18	1.46	1.37	1.64	1.54	1.82	1.73	2.00
30,000	.95	1.17	1.09	1.23	1.13	1.33	1.26	1.5	1.4	1.63
33,000	.909	1.09	.97	1.18	1.09	1.28	1.21	1.39	1.33	1.515
40,000	.825	1.03	.88	1.09	.975	1.18	1.08	1.28	1.18	1.38
44,000	.800	.977	.84	1.05	.931	1.12	1.03	1.23	1.12	1.34
50,000	.76	.94	.82	1.0	.88	1.06	.96	1.16	1.04	1.24
55,000	.727	.927	.782	.963	.854	1.04	.927	1.13	1.00	1.182
60,000	.70	.90	.75	.95	.830	1.0	.90	1.09	.967	1.134
66,000	.682	.863	.727	.909	.79	.97	.863	1.03	.909	1.09

(a) Copper conductors larger than 0.300 in. diameter.

(b) Copper conductors smaller than 0.300 in. diameter.

Note.—Ice loading is not taken into account in the above values. For equal sag and voltage, add 25 per cent. to the above coefficients for : (1) the spacing of copper conductors strung on suspension insulators ; (2) for aluminium conductors.

Example.—It is desired to find the *minimum* spacing of conductors for a 33,000-volt three-phase line, with a conductor sag of 8 ft., using a 0.324 in. diameter copper conductor strung on suspension insulators (delta arrangement). For a conductor strung on pin-type insulators, we have *

$$S = 0.001 \times 33,000 \times 1.09 = 36 \text{ in. ;}$$

add 25 per cent. for conductors strung on suspension insulators is 45 in. For a 44,000-volt line, we have

$$S = 0.001 \times 44,000 \times 0.931 = 41 \text{ in. (pin-type insulators) ;}$$

add 25 per cent., we have

$$41 \times 1.25 = 51 \text{ in. spacing (suspension type insulators).}$$

Also, using the above approximate rule, we have *

$$S = 10 + \frac{.8 \times 33,000}{1,000} = 36.4 \text{ in. (copper) ;}$$

add 25 per cent.

$$= 1.25 \times 36.4 = 45.5 \text{ in. (aluminium).}$$

* These formulæ are for *vertical* spacing of line conductors and wires ; for *horizontal* spacing of conductors, multiply S spacing by 1.15.

Formulae S values for *minimum* spacing of conductors in terms of line-voltages and conductor-sags agree closely with good practice. For values of S given in the chart, an allowance is made for voltages above 11,000 volts; for instance, we have S values for an 11,000-volt line of 19.8 in., but from the formula, $S = 0.001 \times 11,000 \times 1.8 = 19.8$ in. However, from the chart, for the 22,000-volt line $S = 28$ in., and from the formula $S = 26$ in.; and for a 40,000-volt line $S = 39$ in., whereas from formula $S = 35.2$ in.; thus, with increasing voltage the values of S are smaller when derived from the formula.

For the 33,000-volt line (example given at the end of this text) let span in feet = 400; weight (vertical load) in lbs. per feet run of 0.324 in. diameter copper wire = 0.318, but horizontal wind load on the wire at 1 lb. per sq. ft. is 0.027, or 0.678 lb. per sq. ft. at a horizontal wind load of 25 lbs.; for a wind load of 8.0 lbs. per sq. ft. *plus* $\frac{1}{2}$ in. ice on conductor, the load is 0.88 lb. per sq. ft. on projected area of conductor. The *maximum* breaking strength is taken at 60,000 lbs., and the allowable strength at about 2,110 lbs.* for this size of wire.

Therefore, for an absolute minimum conductor spacing $S = 0.001 \times 33,000 \times 1.39 = 45.8$ in., and for an outside maximum spacing $S = 0.001 \times 33,000 \times 1.51 = 50.0$ in. A conductor spacing of 46 in. is decided upon, as will be seen on referring to the example (see p. 253 for vertical arrangement of conductors).

As sags for the ordinary *wood* pole line will range between 2 to 8 ft., the three curves given in the chart should be found useful, and the conductor-spacing for the different line-voltages can be seen at a glance; the values of spacing thus obtained (which are for copper conductors larger than 0.300 in. diameter) can be drawn to conform with good practice. If desired, curves can be plotted on the chart for different standard voltages in terms of S and sag g in inches or in feet, as required; such curves will generally be found more useful than the formula itself, as the values of line-voltage, sag, and spacing of conductors can all be visualised at a glance. Thus, in terms of sag, spacing, and size of line-conductors, line-voltage $E = 1,000(S/K)$ volts. In order to plot curve, assume, for example, that E is 30,000 volts, d is 0.324, then for a 6-ft. sag, $S = 0.001 \times 30,000 \times 1.09 = 32.7$ in., for 8-ft. sag, $S = 34$ in., for a 10-ft. sag, $S = 38$ in., and so on; then

* The allowable strength in lbs. will depend on the factor of safety applied. The ultimate strength of this size of copper wire is around 4,500 lbs. With a wind pressure of 8.0 lbs. per sq. ft. (about 75 miles wind-velocity per hour) and $\frac{1}{2}$ in. radial ice-coating, the total transverse load or the *net* safe transverse load strength of line-support is $3 \times .88 \times 400 = 1,056$ lbs. With a factor of safety of 2.0 the total resultant load (or allowable tension), due to the span and conductors, is $1,056 \times 2 = 2,112$ lbs. The sag increases with increasing temperature, but the tension on the conductor is reduced; with no wind the sag is inversely proportional to the tension.

draw a curve for E through these values of sag and spacing in inches. Or, as desired, for the sags given in the chart, we may plot them as ordinates (in inches) in terms of S in inches (already given) for the different standard voltages shown on the left side of the chart.

Example.—Take a 30,000-volt three-phase line with conductors regularly spaced at the corners of an equilateral triangle (the basis for reactance values given in the chart), with copper conductors each of 0.324 in. diameter (No. 1/0) and sag of 6 ft. The vertical spacing is seen to be 33 in. Also, take the case of a 6,600-volt three-phase line with the three conductors similarly arranged, each of 0.232 in. diameter (No. 4 S.W.G.); sag of conductors is 6 ft. In this case the spacing is seen to be 18 in.

By use of the rough rule, $10 + kE$, for inches we get $10 + 30 = 40$ in. and $10 + 6.6 = 16.6$ in. respectively. And for the rule, $1 + (0.1 \text{ kV.})$ in feet we get $1 + (0.1 \times 30) = 4 \text{ ft.} = 48 \text{ in.}$; and for $1 + (0.1 \times 6.6) = 1.66 \text{ ft.} = 20 \text{ in.}$

In practice, it is usual to transpose the conductors on a three-phase line so that the voltage drop due to reactance shall be the same for different phases. The chart values for reactance per mile of conductor is based upon the formula—

$$2\pi f \left(80.5 + 741 \log_{10} \frac{S}{r} \right) 10^{-8} \text{ ohms.}$$

This formula is correct only for a three-phase line with the conductors regularly spaced at the corners of an equilateral triangle. When the conductors are placed on the same plane (on the same cross-arm), the self-inductance of each of the outside conductors is greater than that of the middle conductor. The average reactance per conductor for a three-phase irregular flat or triangular spacing, where a , b , and c are respectively the irregular spacings, is calculated from the formula $S' = \sqrt[3]{abc}$. For a three-phase regular flat spacing, where the conductors are placed on the same cross-arm with two equal spacings between the middle and the two outside conductors (or placed one above the other), the average reactance per conductor is calculated from the formula $S' = 1.26a$; i.e., $1.26 \times 40 = 50.4$ in.

Example.—Determine the effective spacing S for a three-phase line with conductors supported on one cross-arm and at irregular distances, $a = 40$ in., $b = 40$ in., and $c = 80$ in.

$$S = \sqrt[3]{abc} = \sqrt[3]{40 \times 40 \times 80} = \sqrt[3]{128000} = 50.4 \text{ in.}$$

To keep the voltage drop low the spacing of the line conductors should be as close as practicable. The spacings given in the chart should not be made less than the sags and voltages mentioned.

In the determination of total volts drop for any alternating-current overhead power transmission line (up to the maximum

working voltages now operating in this country) the formula usually involves the expression :—

$$e = I/e',$$

where I = current in amperes for average load in kW.

l = transmission distance or length of line in miles.

$$e' = R \cos \phi + X \sin \phi.$$

Now, if we divide this formula into two others, one of which represents the reactance, by

$$2\pi f L' = 2\pi f [0.08 + 0.741 \log_{10} (S/r)]$$

and the other formula which represents a certain geometric difference, then, if we express these two formulæ in function of the same independent variable, such as the diameter d of conductor or some other quantity which contains it, the intersection of the two geometric lines will give the diameter or the value of the variable that contains it. Values may be plotted so that the first quantity represents the reactance corresponding to S/d and f ; the second quantity to represent a system of parabolas which will depend on

$$(e''/\cos \phi)d^2 \text{ and } (\sin \phi/e'')f,$$

where

$$e'' = (E_s - E_r) \div (lI),$$

in which

$$E_s = E_r + e; \text{ where } e = (E_s - E_r) \div E_r.$$

The two common formulæ used for calculating voltage drop are :

$$(\text{three-phase}) \sqrt{3}I(R \cos \phi) + (X \sin \phi)l$$

and

$$(\text{single-phase}) 2I(R \cos \phi) + (X \sin \phi)l.$$

The two variables and unknown constants (R and X) can, for any spacing, frequency, and size of conductor, be obtained from the chart presented herein. For a three-phase line the value of I is obtained from the formula

$$I = P \div (\sqrt{3}E_r \cos \phi) = 0.5773[(P' \div E_r \cos \phi)].$$

Much greater simplicity can be obtained where the ratio of conductor-spacing is plotted to logarithmic scale and the reactance plotted to arithmetic scale, values of the different frequencies forming straight lines, respectively. The accompanying chart is drawn up along these lines. The line inductive-reactance is constant for fixed values of the ratio $S/d = S/2r$, and the minimum value of line reactance will occur when the conductor values have zero separation, *i.e.*, for the ratio $S/(d/2) = 2.0$. For the minimum size of copper wire recommended, which may be taken as a No. 7 (0.160 in. diameter) or No. 8 S.W.G. (0.176 in. diameter), depending on the span, climatic conditions, etc., the ratio of S/d is 150 for 24 in., 225 for

36 in., 300 for 48 in., 375 for 60 in., and 600 for 96 in. spacings. For the maximum size of copper wire, which may be taken as No. 7/0 S.W.G. (0.500 in. diameter), the ratio of S/d is 48 for 24 in., 72 for 36 in., 96 for 48 in., 120 for 60 in., and 192 for 96 in. spacing, respectively. The accompanying chart gives ratio of S/d going much below and much higher than these values.

The accompanying chart has been plotted on the basis of the ratio S/d for different values of f (standard values among others). Values of ohmic resistance R for copper, aluminium, and steel-aluminium conductor are also given on the same chart. Values for vertical conductor-spacing in terms of line-voltage and sag (2 to 8 ft.) are also given. For direct practical use, the chart is represented in values of reactance-volts per ampere (IX) and resistance-volts (IR) per ampere per mile of single conductor, respectively. If it is considered preferable to include the *standard* sizes of conductors in place of the diameter d , all that is necessary is to look up a manufacturer's wire table, such as the one given herein, and draw a vertical line through the corresponding value for d ; the present commercial sizes are not given because they are likely to change, whereas the value of d is constant and fixed *always*. They are, however, given in the table opposite.

By some engineers it may be considered more convenient to include the *standard* commercial sizes of conductor in the chart. This is purposely omitted at the expense of giving simplicity to the chart. However, all that is necessary is to mark in perpendicular lines corresponding to the respective diameters given in the above table which shows the diameter, area, size, and weight of *bare*, hard-drawn copper (solid and stranded) conductors. Starting from $d = .324$ in. diameter (No. 1/0 S.W.G.) at the top left-half of the chart, a perpendicular line is drawn in as a guide for the marking in of all the other standard sizes, if required. In order to avoid any possibility of making the chart look complicated, the horizontal values for line-voltage E and reactance-volts IR (plotted to logarithmic scale, respectively) are not drawn across the chart but are shown on the left side of the chart. In its present state this chart is of international use, as both American and Continental standard conductor sizes can be drawn in as desired—British and American standards in particular, as these standards obviate metric equivalents.

The corresponding sine values for the assumed values of cosine (power factor) can be obtained from the accompanying chart, which gives a range of the values of sines, cosines, tangents, and the angle ϕ commonly used. From the accompanying chart we can promptly find the voltage drop in any *alternating-current* circuit for any frequency employed in power transmission work and for any value

TABLE XXII.—STANDARD SIZES OF COPPER CONDUCTORS

*Giving Diameter, Area, Size, and Weight per Mile (in lbs.) of
Hard-Drawn "Aerial" Copper Conductors*

STRANDED			
Diameter of Cable in Inches.	Area in Square Inches.	Wire Strands and Diameters.	Weight per Mile in Lbs.
.770	.350	37/.410	7,295
.707	.300	37/.101	6,149
.644	.250	37/.092	5,104
.574	.200	37/.082	4,052
.504	.150	37/.072	3,109
.505	.150	19/.101	3,156
.460	.125	19/.092	2,619
.410	.100	19/.082	2,081
.360	.075	19/.072	1,603
.290	.050	19/.058	1,040
.285	.050	7/.095	1,028
.204	.025	7/.068	526
SOLID			
Diameter of Wire in Inches.	Area in Square Inches	Size of Wire (S.W.G.)	Weight per Mile in Lbs.
.500	.1964	No. 7/0	4,010
.464	.1691	6/0	3,464
.432	.1466	5/0	2,984
.400	.1256	4/0	2,558
.372	.1087	3/0	2,211
.348	.0955	2/0	1,935
.324	.0824	1/0	1,678
.300	.0707	1	1,438
.276	.0598	2	1,217
.252	.0498	3	1,015
.232	.0423	4	860
.212	.0353	5	718
.192	.0289	6	589
.176	.0243	7	495
.160	.0201	8	409
.144	.0163	9	331

of the ratio S/d , *i.e.*, any value of the diameter d and any spacing S ranging from about $\frac{1}{2}$ in. up to conductor-spacings much greater than 180 in. Therefore, the chart can be used with equal advantage for underground cables or insulated cables in general where capacitance is not involved. The value of R would thus be given the usual 2 per cent. allowance for *annealed* copper. We can also obtain any one of the four variables (d , S , f , and X) when any three are given. The upper lines for reactance-volts value in terms of f are (in each case) for stranded conductors and the lower lines for solid conductors. This is due to the fact that the cross-sectional area of a stranded conductor has a greater diameter than the equivalent cross-sectional area of a solid conductor. Thus, for equal diameter d a cable will have a greater reactance-volts drop and a greater resistance-volts drop than a wire; in other words, the values of reactance and resistance are the same for equal cross-sectional area of cable and wire.

Let us now consider a rather complete problem involving a line-voltage (a receiving voltage of 30,000 volts) which, with the present state of the art of overhead power transmission work, is about the most reliable, also about the most economical from the viewpoint of total line cost and total terminal apparatus cost, and a voltage that should have a wide field of usefulness in this country, chiefly because of the wide gap between this voltage and the next higher *standard* voltage, *i.e.*, 60,000 volts.

Example.—A three-phase power transmission line, requiring 30 miles of overhead line, is to be constructed from a hydro-electric generating station to a town which has its own local steam plant. The estimated future three-phase full load is 3,000 kW. and the power factor of the load for designing purposes is taken at 80 per cent. lagging; the frequency is 50 cycles per second. Given the size of hard-drawn copper wire is .324 in. diameter (No. 1/0 S.W.G.), determine the following: (a) line-voltage; (b) span, sag, and line-support strength; (c) spacing of conductors; (d) line-current; (e) power loss; and (f) voltage-regulation.

Line-Supports.—According to standard specifications and quotations received from manufacturers, it is decided to use a steel pole of the lattice type, similar to that shown in Fig. 30, with the exception of insulators and cross-arms. Both cost and greater span-length are the decisive factors in this choice, as well as the relative life of wood and steel.

Instead of using an earthed wire at the top of each pole, a slightly better insulator is used. It is estimated that the use of an earthed wire would increase the transverse pole load about 25 per cent., and would require a closer pole spacing or a stronger pole.

The *average* span is estimated at 400 ft. ; the estimated absolute minimum sag 7.5 ft. and maximum sag 11 ft.

The *effective* length of pole required (not the over-all length) is estimated at 40.0 ft. ; each pole is set 5.5 ft. into the ground.

The estimated *net* safetransverse load strength required is 1,060 lbs. for straight runs.* There are no angles in the line greater than 15° , and to provide for the extra strength at these points stay-wires are used, as well as a larger sized pole. The size of the stay-wires is calculated by taking the total working stress of a .324-in. diameter hard-drawn copper wire at 2,260 lbs. ; thus the total dead-end stress due to the three line-conductors will be $3 \times 2,260 = 6,780$ lbs. The resultant horizontal stress on the line-supports due to an angle of 15° in the line is $2 \times 6,780 \times \sin (15/.2) = 2 \times 6,780 \times .1306 = 177$ lbs. The resultant angle at the line-support is taken at 30° , hence the total stress in the stay is $177/\sin 30^\circ = 177/.5 = 354$ lbs. Then, for factor of safety of 3, the ultimate strength required of the stay-wire is $3 \times 354 = 1,062$ lbs. A $\frac{5}{16}$ -in. diameter steel-stranded cable will be satisfactory, or two .165-in. diameter telegraph wires can be used for the stay-wire.

Minimum spacing of conductors is estimated at 3.6 ft. (42 in.). From Table XXI. we find spacing for 33,000-volt line and 11-ft. sag to be 42 in. for conductors regularly spaced at the corners of an equilateral triangle ; a spacing of 46 in. is decided upon for the 400-ft. span ; for this sag a 42-in. spacing would serve quite well. In practice it is the maximum sag which decides the minimum spacing of conductors ; this can be obtained from the above table by taking 11.0 ft. maximum sag.

Size of Conductor.—As a *first trial* this is assumed as 0.324 in. diameter (No. 1/0 S.W.G.) hard-drawn solid copper wire. From the chart for IR we find 0.55 resistance-volts drop per ampere per mile of single conductor, making a total of $.55 \times 30 = 16.5$.

Line-Voltage.—As a *first trial* this can be taken at 33,000 volts at the generating end. That is, for a load of 3,000 kW. at 80 per cent. power factor, or $3,000/.8 = 3,750$ kVA., we have, line-voltage

$$E = (1.05 + .00001 \times 3,750) \times 30 = 32.6 \text{ KV.}$$

The nearest standard receiving voltage E, is taken at 30,000 volts.

Power Factor.—In this new line, as in nearly every new line construction, the power factor cannot be accurately determined ; in such cases it is usual to assume 80 per cent. for a motor load and 85 per cent. for a mixed motor and lighting load. For designing purposes the former condition is taken.

* The actual loading *on the conductor* is the resultant of its weight (*w*) and the wind pressure (*W*), thus,

Line-Current.—Full load (three-phase) line-current is

$$I = .58 \times \frac{3000 \times 1000}{30,000 \times 0.80} = 72.5 \text{ amperes.}$$

Total Power Loss.—Power loss in the three-phase line (one three-wire circuit *only*) is

$$3 \times (72.5)^2 \times 16.5 = 260 \text{ kW.},$$

and the percentage power loss is

$$(260/3,000) \times 100 = 8.67 \text{ per cent.}$$

Conversely, if we assume that the percentage power loss is given and not the size of conductor, then for 8.67 per cent. power loss we can determine size of conductor from the chart. Thus total power loss is

$$(8.67/100) \times 3,000 = 260 \text{ kW.}$$

Load current is 72.5 amperes (as before); then ohmic resistance for 30 miles of conductor is

$$(1/3) \times (260,000/72.5^2) = 16.5 \text{ ohms,}$$

and resistance per mile of conductor is $16.5/30 = 0.55$ ohm. From the chart, size of hard-drawn copper wire is seen to be 0.324 in. diameter (No. 1/0 S.W.G.).

Voltage-Regulation.—From the chart, R at $25^\circ \text{C.} = .55$ ohm per mile, and X at 50 cycles = .598 ohm per mile; that is, $S/d = 141.9$, making $IX = .598$ volt per ampere per mile.

$$e = \sqrt{3} \times 72.5 \times (.55 \times .8) + (.598 \times .6) \times 30 = 3,000 \text{ volts.}$$

$$E_k = E_r + e = 30,000 + 3,000 = 33,000 \text{ volts,}$$

and percentage voltage-regulation is

$$[(33,000 - 30,000)/30,000] \times 100 = 10.0 \text{ per cent.}$$

CHAPTER XIV

OVERHEAD ELECTRIC POWER TRANSMISSION ECONOMICS

It is sometimes found that a decision as to the system, frequency, and/or the line-voltage is dependent in some way or other on electric power systems already operating within the area of supply or within transmission distance. As regards the line distance limit, economy *only* is the dictator and not the distance of transmission. Usually, where hydro-electric power is involved, the distance of transmission as also the maximum kVA. capacity are defined rather by the natural sites and conditions offered, the sale price of energy, and the kVA. yearly demand.

The economic problem of electric power transmission involves several independent variables, principal among which are: the distance of transmission ; the amount and nature of load ; the price of delivered energy ; the generating cost of energy ; yearly expenses on the investment ; the system, frequency, number of circuits, and the power factor ; size, spacing, and arrangement of the line-conductors ; span-length of line-supports ; the factors of safety, and so forth. Of these, the most important variables which govern the most economical design are : (a) the line-voltage ; (b) the line-conductors ; (c) the spacing of line-supports, and for a.c. systems ; (d) phase-compensation. As regards the latter variable, this is economical when the efficiency of transmission is improved, and the kVA. yearly expense is decreased thereby. The power factor ($\cos \phi$) has a direct influence on the cost of transmission, but it has a less influence on the most economical voltage, the most economical size of conductors, and the most economical span of line-supports. Voltage regulation and the efficiency of large power transmission systems can generally be improved by installing synchronous condensers at the receiving end of the line.

To know whether a given transmission system is commercially possible or not, it is first necessary to know the generating cost and the sale price and amount of delivered energy. In the determination of the most economical conditions it is necessary to establish the relation between the different parts of the transmission system and the variable factors above-mentioned. This is no easy task where

great accuracy is required ; as an instance, the relations may be (and usually are) established for a fixed set of conditions, also a fixed value of the *constants* contained in them, which should be determined or assumed from actual practical information to best suit the given conditions, locality, etc. In general, the conditions assumed, as also the relations and their constants, are, in actual practice, subject to radical changes ; for example, financial and/or industrial conditions may alone change the value of energy as well as the cost of the transmission system. The constants given in the different calculations for a determination of the most economical conditions must differ for the various conditions commonly met with in actual practice ; that is to say, the values will be different under different market conditions and localities, temperature changes, etc., hence they should be chosen with great care based on an intimate knowledge as well as a broad practical experience of the actual work to be done.

The yearly cost of energy lost in the transmission system and the yearly expenses (interest, depreciation, taxes, etc., on the total capital invested) should make up the difference between the cost of generating energy and the sale price of the energy per kVA.-year. The transmission system cost which involves the sale price of energy and the total quantity of energy delivered, may be expressed in the form :

$$C_t = C_g \left(\frac{P}{e} \right) + kK.$$

That part of the cost due to energy loss in the transmission system may take the form : $C_y = (I/e) - I$, and the investment cost of the transmission system may take the form $C_x(K/P \cos \phi)$, wherein $C_x = k/C_g$. Therefore, the kVA.-year transmission system cost is

$$C_t = C_x \left(\frac{K}{P \cos \phi} \right) + C_y,$$

where

P = kVA.-year of delivered energy. The maximum yearly amount of energy transmitted in kW.-hours is kW. $\times$ load factor $\times 8,760$; for 100 per cent. load factor, kW.-year is $(P \cos \phi) \times 8,760$. In an a.c. system there is generally a difference in phase between the current I and the voltage E producing a lagging power factor $\cos \phi$; this latter must be taken into account, as it bears a direct influence on the cost of transmission.

K = the amount of money invested or total capital cost of the system.

C_g = cost of generating the energy per kVA.-year.

k = yearly expenses ; made up of the total interest, depreciation, taxes, etc., on the total investment. The total percentage

allowance for these yearly expenses will not be the same for the different parts of the transmission system.

$\cos \phi$ = average power factor of the transmission system. The cost is practically inversely proportional to the power factor.

e = transmission system efficiency, *i.e.*, ratio of the receiver load to that generated at the sending end. High efficiency means less loss ; on the other hand, it generally means a more costly system and sometimes a more reliable system and one offering better voltage regulation. That is to say, we may apply better factors of safety, install duplicate transmission lines and/or units in the stations, better lightning protection equipment, and install synchronous condensers for phase-compensation, etc.

The total energy loss would, ordinarily, be given in terms of load conditions at the receiving end of the line. The total loss may also be subject to corona and leakage, etc. The load at the generating end would represent the sum of the total losses and the receiver load. A decrease in energy loss in the transmission system, or a decrease in the item $(I/e) - I$, which is the same thing as an increase in e , can be brought about by increasing K (provided the extra investment is made in the proper direction), or it can be made by decreasing P ; the objective in any case is that of determining the most economical conditions. The total cost items would be made up of the ohmic and the magnetic losses and, perhaps, the charging-current (electrostatic), corona, and leakage in the transmission lines and the transformers, as well as the yearly expenses of line-conductors, line-supports, insulators, transformers, and all the other items required to make up the complete transmission system. The transmission cost due to the energy loss would ordinarily take the form :

$$C_p = R_c + T_c + T_m,$$

where

C_p = cost due to the energy loss in the transmission system.

R_c = cost due to the ohmic loss in the line-conductors.

T_c = cost due to the sum of copper losses in the transformers.

T_m = cost due to the sum of core losses in the transformers.

This energy loss is variable according to the variation of power and power factor, the number of transformers in service, time-period in service, etc., and the total energy loss in kVA.-year is dependent on the ohmic loss in the line-conductors, copper and magnetic losses in the transformers, and the load factor of the transmission system. The ohmic loss in the conductors depends on the amount and power

factor of load, and on the size and length of the line-conductors, etc. The ohmic loss in the transformers depends *only* on the amount and nature of the load. The magnetic loss in the transformers is practically independent of the load.

In arriving at the cost of a transmission system it would be better to divide the system into two parts, one consisting of the transmission line which is made up of the line-conductors, line-supports (complete with struts or stays), insulators, earth wires, telephone wires, and wayleaves, etc.; the other part consisting of the various stations costs, made up of transformers, switchgear complete, accessories, lightning protection equipment, buildings and land, and so forth. The former cost is practically proportional to the distance of transmission, whereas the latter cost is practically independent of the distance of transmission, hence the reason for dividing the system cost into two parts. In many cases the percentage cost of the line-conductors or of the line-supports will represent practically double the combined cost of insulators, telephone circuit, and wayleaves. For first-class quality of materials, construction, and layout, the percentage cost of transformers, switchgear complete, and buildings with land will, for extra high voltage systems, be about equally divided. In this text it is intended to treat mainly with the cost of that part of the economic problem representing the transmission line proper, including the power transformers at each end of the line.

The kVA. capacity of a transmission line is directly proportional to the cross-section of the line-conductors and inversely proportional to the yearly cost. With regard to the total cost of the entire transmission system this cost will increase with the increase of transmission distance. The line-voltage will increase with an increase in kVA. capacity, the distance of transmission, the cost of energy, and the load factor, and with a decrease in the yearly expenses (interest, depreciation, taxes, etc., on the total investment).

The cost of line-conductors is proportional to their length and their cross-section. This cost may be expressed :

$$C_c = NlA$$

where C_c = cost of a given area of conductor.

l = length of conductor (one way).

A = cross-section of conductor.

N = number of conductors.

The size of conductors increases with an increase of kVA. capacity, with an increase in the cost of energy, and an increase in the load factor. When the load factor is high, the ohmic loss will tend to

increase, and the yearly expense on the investment decrease, resulting in a high line-voltage and large size of conductors.

The cost of a line-support is composed of the support itself (with or without stays or struts), its foundations, land, wayleave or easement, and the erection cost. The total cost will vary according to the market conditions, line-voltage, the allowable stresses, angle of the line, span-length, the locality, climate and temperature. The yearly cost will be high when the span of line-supports is too long; also, the yearly cost will be high when the span of line supports is too short for certain sizes of conductor.

The cost of a line-support increases directly with the increase in vertical and transverse loads, etc. The cost may be expressed :

$$C_s = N(c + F + L + H),$$

where C_s = cost of line-supports.

c = cost of transport and haulage.

F = cost for erection and foundation.

L = cost of structure or pole.

H = cost of hardware and cross-arms.

N = number of line-supports completely fitted.

The cost of land, wayleave, or easement is not taken into account, as these items should be treated independently.

The height of a line-support depends on the line-voltage E , locality, span, sag, temperature-range, and the minimum height allowable by law. A line-support is high when the line-voltage is high and the span and sag great. There is a minimum height independent of line-voltage E , span, sag, and the separation of the line-conductors; the effective height involves these conditions and may be expressed $h_m + xE + ys^2$, where E = line-voltage, s^2 = square of span-length between line-supports, h_m = minimum height independent of E and s , x = a constant, y = another constant. With regard to such variables as line-voltage, sag of line-conductors, and span of line-supports, the cost of a line-support (steel, for instance) has to be considered in the light of the different kinds of mechanical stresses, such as the weight of the structure itself, windage, the vertical, longitudinal, and the transversal loads, and other factors which influence the total weight and cost, as well as the grade and cost of foundations and erection. The most economical span of line-supports is defined not only by this cost but also by the cost of the line-insulators, etc.

The cost of an insulator is dependent on the type and size or weight; the latter is dependent on the line-voltage E ; it is also dependent on the insulating capacity which defines the cost. For

the pin-type of insulator the cost is composed of the porcelain body and the pin ; the cost may be expressed

$$C_i = C_{x''} + C_{y'''} = C_{i''}$$

where C_i = total cost of the insulator.

$C_{x''}$ = cost of porcelain body and pin.

$C_{y''}$ = cost of pin.

$C_{y'''} =$ cost of porcelain body.

And $C_{x''}N$ will represent the total cost of insulators.

Where there are a number of porcelain bodies, metal pins, and caps, as in the case of a suspension type insulator, $C_i = N'C_{x''}$, where N' = number of units. The cost may be expressed in terms of either unit voltage of the string or the total line-voltage for the complete string of units.

The cost of a transformer is made up of copper, iron, and insulating materials. The cost due to kVA. capacity consists of copper and iron. The cost of insulation is dependent on the total area of copper surface and the depth or thickness of insulation. The cost of insulation is usually small as compared with copper and iron ; it is proportional to the voltage and the square root of the kVA. capacity, and is independent of the induced voltage (per turn of the windings), but the induced voltage is proportional to the square root of the kVA. capacity. When the kVA. capacity is great the induced voltage must also be great to satisfy the most economical conditions. The total increase in cost by an increase in voltage E is different in magnitude for a different kVA. capacity and voltage. It is therefore reasonable to assume a condition so that a factor $x(P)$ of a transformer is made up of that part of the cost which is independent of the line-voltage E , and a factor $y(EP)$ of a transformer is that part of the cost which is added by the increase of voltage E , and which is nearly all dependent on E but very little on the kVA. capacity of the transformer. The cost of a transformer may therefore take the form :

$$C_T = x(P) + y(EP),$$

where

C_T = total cost of a transformer.

P = kVA. capacity of the transformer.

E = average line-voltage.

$x(P)$ = a factor representing that cost which is independent of the voltage E .

$y(EP)$ = a factor representing that cost which is added to the increase in voltage.

Then C_TN will represent the total cost of transformers ; wherein N = number of units.

The cost of wayleaves, telephone circuit, earth wire, and other items of cost which can be classed as miscellaneous to the power transmission line proper, will more often increase with the kVA. capacity of transmission, and increase directly proportional to the transmission distance. This cost may therefore be expressed

$$C_m = x(l + yP)l,$$

where l = distance of transmission.

P = kVA. capacity of transmission.

x, y = constants.

It is evident from the above-mentioned, that, with the exception of the transformers and h.t. switchgear, the cost of the whole of that part of the transmission system which is practically proportional to the distance of transmission can be expressed :

$$C = C_c + C_s + C_i + C_m \dots,$$

where C_c = cost of given size of conductors for a given system.

C_s = cost of given line-support, its foundation and erection.

C_i = cost of the line-insulators complete.

C_m = cost of wayleaves and miscellaneous.

The life of the various parts of a transmission system depends on a number of diverse conditions, the principal among which are : kind and quality of materials and workmanship, climatic conditions, chemical action, class and amount of protective equipment used, grade of handling during transit, and its installation and the severity of electrical and mechanical stresses in operation. Under reasonably good conditions and for good quality materials, we may take the life in years and the percentage yearly charges of the whole of that part of the transmission system (the cost of which is proportional to the distance of transmission) to be fairly representative of the following mentioned :—

From the viewpoint of cost of energy losses in transmission and yearly expenses on the capital invested, etc., a summary of some of the more important transmission conditions shows that :—

For a definite kVA. capacity, line-voltage, and distance of transmission, the yearly expense of ohmic loss in the line-conductors will decrease with an increase in the size of conductors, and vice versa.

For a definite generating cost of energy, distance of transmission, and total kVA. capacity, there is a minimum cost for a definite line-voltage and certain size of conductor.

For a definite kVA. capacity, generating cost of energy per kVA.-year, distance of transmission, and line-voltage, the minimum cost

TABLE XXIII.—SHOWING INTEREST, DEPRECIATION, AND MAINTENANCE VALUES FOR DIFFERENT ITEMS OF COSTS, ETC.

Part of Transmission.	Life (L) in Years.	Interest (i') in Percent-age.	Taxes (t') in Percent-age.	Depreciation (d') in Percent-age.	Maintenance (m') in Percent-age.	Yearly Expenses (k) = $i' + t' + d' + m'$
Conductors—						
Copper — —	20	6	.9	3.187	2.5496	12.637
" — —	25	6	.9	2.245	1.796	10.941
Line supports—						
Wood — —	16	6	.9	4.401	3.521	14.822
" — —	20	6	.9	3.187	2.5496	12.637
Steel tubular poles	20	6	.9	3.187	2.5496	12.637
" — —	25	6	.9	2.245	1.796	10.941
Lattice poles —	35	6	.9	1.227	0.9816	9.109
" — —	40	6	.9	0.934	0.7472	8.581
Towers (wide base)	20	6	.9	3.187	2.5496	12.637
" — —	25	6	.9	2.245	1.796	10.941
Insulators—						
Porcelain — —	9	6	.9	9.257	7.4056	23.663
" — —	12	6	.9	6.467	5.1736	18.541
Miscellaneous—						
Earth wire —	12	6	.9	6.467	5.1736	18.541
" — —	16	6	.9	4.401	3.521	14.822
Telephone wires	16	6	.9	4.401	3.521	14.822
" — —	20	6	.9	3.187	2.5496	12.637
Yearly expenses for that part of the transmission system which is independent of the distance of transmission but which represents a part of the high-tension system, are given as :						
Transformers —	17	6	.9	4.042	3.2336	14.176
" — —	25	6	.9	2.245	1.796	10.941
SW-gear (h.t.) —	20	6	.9	3.187	2.5496	12.637
" — —	30	6	.9	1.639	1.312	9.850

(L) As a general rule there is a range in years of life* for a given quality of material due to many causes such as those mentioned above; the duration period-range given here for practical use is considered to be reasonable.

* The range given here should not be averaged; moreover, there will certainly be cases where the life will be much less, as also cases where the life in years will be much more.

(i') This is often taken at 6 per cent. throughout.

(t') This is taken at a fixed value of 0.9 per cent.; the different taxes will vary for the different countries and localities.

(d') This is equivalent to a sinking fund which will provide a sum of money sufficient to pay off an amount at the end of a given period of years; the rate of interest is taken at 4.5 per cent. Engineers are at variance as to a proper value for each item of cost (see *Electrical Review*, August 1924).

(m') This is taken at a fixed value (for each part of the transmission) of 80 per cent. of the depreciation (d').

(k) This represents the total yearly expenses, and is made up of the sum of the interest, depreciation, and taxes, etc.

of transmission will decrease for an increase in the size of conductors up to a certain cross-sectional area.

For a definite kVA. capacity and transmission distance, an increase in the generating cost of energy per kVA.-year for minimum cost of transmission will result in a larger size of conductor for a given transmission voltage.

For a definite transmission distance and kVA. capacity and a given line-voltage, the size of conductors will increase with an increase in the generating cost of energy per kVA.-year.

For a definite kVA. capacity, transmission distance, and generating cost of energy per kVA.-year, an increase in line-voltage will result in a decrease in the size of conductors.

For a given percentage power loss, transmission distance, and line-voltage, the yearly charges per kW. on all the terminal apparatus and equipment increases more slowly than the power output of the system.

For a given line-voltage the effective height of line-support will decrease with a decrease in span-length, and vice versa.

For a given span of line-supports the vertical and transverse loads will decrease with a decrease in the size of conductors, and vice versa.

For a given span of line-supports, the effective height of the line-supports must be increased for an increase in line-voltage.

With an increase in span of line-supports and definite size of line-conductors, the vertical and transverse loads will increase.

For a definite size of conductors, line-voltage, and transmission distance, there is a minimum yearly expense for a certain span of the line-supports.

For a definite span of line-supports, line-voltage, and transmission distance, the yearly cost of line-supports will increase with an increase in the size of conductors.

For an increase in kVA. output the size of conductor must be increased in the same proportion, so that the cost per kW. due to the line-supports and hardware still increases with the transmission distance.

For a definite power in kVA., transmission distance, and generating cost of energy per kVA.-year, the minimum cost for a given span of line-supports will increase with an increase in the size of conductors, but the increase in cost will be less for a lower line-voltage and shorter span of line-supports.

For a definite cross-sectional area of conductor and generating cost of energy per kVA.-year, there is a minimum cost of transmission for a certain span of the line-supports.

For a definite yearly generating cost of energy, transmission

distance, and total kVA. capacity, there is a minimum cost for a certain line-voltage.

For a definite span of line-supports the cost of line-insulators will increase with an increase in line-voltage. And for any line-voltage the cost will increase with a decrease in span of line-supports.

For the pin-type of insulator the cost will increase with the rated voltage of the insulator.

For the suspension-type of insulator the cost curve will be similar in form to the pin-type. However, the rated voltage and the number of insulator-units forming the string will increase with the line-voltage. For a given line-voltage the cost is a minimum for a certain size and number of insulator-units.

The cost of a transformer will increase with the increase in line-voltage and kVA. capacity. When the kVA. capacity increases the cost will increase due to the increase in the mean lengths of active materials and the cross-section areas. When the voltage increases, the mean length of copper will increase while its cross-sectional area will decrease slightly; but the volume of iron will increase greatly. The losses in the transformers will be practically independent of the line-voltage, transmission distance, and the size of line-conductors.

For a definite kVA. capacity the yearly expenses will decrease with a decrease in line-voltage. Also, for a definite line-voltage, the yearly expense will increase with an increase in the kVA. capacity, and vice versa.

For an increase in the generating cost of energy with a definite transmission distance, span-length, and delivered load, there will result an increase in the size of conductors and line-voltage, but a decrease in the transmission cost per kVA. or kW.-year.

For an increase in the distance of transmission with a definite generating cost of energy, span-length, and delivered load, there will result an increase in the line-voltage transmission cost of energy and the cross-sectional area of conductors.

For an increase in the delivered load with a definite generating cost of energy, span-length, and transmission distance, there will result an increase in the line-voltage and size of conductors, but a decrease in the transmission cost of energy per kW. or kVA.-year.

A long transmission will require a high voltage to satisfy the most economical conditions.

So long as power transmission lines exceed certain limits, the *capacity* current (I_c) will present the most difficult and may be *the* most dominating factor.

If the kVA. capacity and distance are increased greatly, the magnitude of line disturbances will be increased, and the effect of

line inductive reactance may place a limit on the economical transmission distance.

For any given load, transmission distance, and power factor, there is a voltage beyond which any increased efficiency is impossible.

When the transmission distance is increased, the cost due to the ohmic loss in the line-conductors will tend to increase proportionally.

For a *fixed* power output the yearly charges per kW. on the line-conductors will increase with the square of the transmission distance, whilst those on the line-supports will increase directly with the distance (assuming constant transmission voltage and constant ohmic voltage drop or constant percentage power loss).

The first cost of the transmission line-conductors, for transmitting a given amount of power over a given distance with a given power loss, can be reduced only by increasing the transmission line-voltage.

The limiting distance to which power can be transmitted *economically* depends finally upon the cost of the line-conductors.

The yearly capital charges on the complete line *plus* the minimum profit which is considered satisfactory are of first importance in determining the distance to which power can *economically* be transmitted—the less this sum, the greater the distance power can be transmitted.

TABLE XXIV.—WATER-POWER DEVELOPMENTS OF THE LEADING NATIONS OF THE WORLD

Country.	Total Potential h.p.	Total Developed h.p.
Australia — — — — —	750,000	...
Austria — — — — —	3,700,000	280,000
Brazil — — — — —	25,000,000	550,000
Canada — — — — —	26,000,000	3,230,000
Chile — — — — —	2,500,000	60,000
China — — — — —	(Not known)	...
Czechoslovakia — — — — —	1,725,000	150,000
Denmark — — — — —	(Small)	9,000
Dutch East Indies — — — — —	15,000,000	80,000
Estonia — — — — —	170,000	17,000
France — — — — —	6,000,000	1,400,000
Great Britain — — — — —	(Not known)	250,000
Holland — — — — —	None	...
Hungary — — — — —	(Very small)	3,000
Mexico — — — — —	6,000,000	500,000
India — — — — —	7,400,000	340,000
Japan — — — — —	14,100,000	3,050,000
New Guinea and Papua — — — — —	10,000,000	...
New Zealand — — — — —	4,900,000	30,000
Peru — — — — —	4,500,000	40,000
Norway — — — — —	12,300,000	1,300,000
Russia — — — — —	2,000,000	100,000
Siberia — — — — —	51,140,000	90,000
Sweden — — — — —	...	3,420,000
Switzerland — — — — —	8,000,000	1,500,000
Tasmania — — — — —	700,000	70,000
Union of South Africa — — — — —	1,750,000	6,000
United States of America — — — — —	26,000,000	6,780,000

INDEX

ABSOLUTE minimum flow, 4
 Advantage of tying plants in electrical parallel, 4, 5
 Air valves, 123
 Amount of power calculated, 9
 Anchoring of pipe, 121, 123
 Annual costs, 192
 Appointment of Commission, 203
 Atmospheric pressure, 12
 Arrangement of stream flow records, 24
 Automatically operated plants, 163-183
 Average daily rate of flow, 61
 — duration curve, 31
 — discharge, 17
 — quantity of water available, 8
 — rainfall, 17, 29

BACK-WATER conditions, 45, 149
 Balanced piston type needle valves, 15
 Barometric pressure, 11
 Base power, 92, 185
 Basin of a river or stream, 71
 Bearing weight of foundations, 46
 Boring and test pits, 38, 111
 Bottom width of conduit, 97, 101, 105
 Brake horse-power, 13, 42
 Building up of load, 60
 Bulk power consumers, 187
 Butt joints, 132, 135, 141

CAPACITY of pipes, 129
 Capillary action, 73
 Catchment area (drainage area), 16, 29, 64, 72
 Channel conditions, 88, 105
 — of approach, 75
 Chemical gauging, 73, 78, 86
 Choice of water-power sites, 8
 Cippoletti weir, 77
 Circular pipe joints, 126
 Climatic conditions, 27
 Coefficient of roughness, 45, 87, 95, 124
 Common law principles, 204
 Completion of power estimates, 25
 Computing power and energy, 42
 Concrete lining of conduit, 96, 100, 104, 105, 113, 124
 Condition of ground, 17, 73
 Conduit, 93, 158
 — installations, 115
 — location, 31, 38, 93
 — side-slopes, 105

Conservation, 154, 207
 Contours, 48, 54
 Control of water, 204
 Cost of steel pipe, 135
 Cross-section of stream, 71, 80, 83, 86, 105
 Current meter, 73, 81, 85
 Current meter operation, 86
 Cusecs (cubic feet per second), 8

DAMS, 38, 40, 49, 50, 93
 Datum, zero or bench-mark, 70
 Deflecting nozzle, 15
 Dependable flow, 64
 Depreciation, 199, 202
 Depth of water, 98, 100, 105
 Design of power transmission lines, 220, 226, 231
 Design of water-power conduits, 106
 Determination of probable run-off, 24
 Development of the available power, 5
 Diameter of jets, 9
 — of pipes, 130, 131
 — of wheel, 9, 14
 Direct discharge method, 11, 73
 Discharging efficiency of a stream, 71
 Discharge measurements, 80, 95
 — through pipes, 128
 Distinguishing direction of flow, 71
 Diversity factor, 60
 Diversion weir, 39, 40
 Downstream and upstream, 71
 Draft from reservoirs, etc., 41, 62, 69
 — tube, 6, 10, 11, 147, 149, 159, 162
 Drainage area (catchment area), 52
 Drought periods, 143
 Dry weather flow, 72

EARLY hydraulic works, 1, 2, 163
 Earthing the neutral point, 217
 Earth section of conduit, 96, 99, 101, 113
 Economic aspects of electric power transmission, 255-265
 — development, 6, 35
 Economical design of conduit, 125, 146
 — plant capacity, 154
 — pipe section, 125-128
 Eddies and whirls, 99, 101
 Efficiency, 13, 148, 222
 — constants for h.p. and kW., 91
 — of pipe joints, 132, 141
 Effective cross-section, 146

Electric power transmission limit, 209
 Energy from storage, 56
 — loss, 99, 127, 146
 English *common law*, 203
 Estimated annual costs, 192
 Estimate of costs, 197
 η values for different conditions of channel,
 87, 145
 Evaporation, 22, 26, 101, 143, 201
 Examinations, 37
 Examination of the annual rings of trees, 18,
 143
 Expansion joints, 122
 — of steel, 123
 Extreme minimum stream flow, 35
 Excess storage, 153

FACTOR of safety, 129, 141
 Favourable conduit section, 105
 Float method of measuring flow, 85
 Flood water, 60, 64, 72
 Flow measurements, 73, 78
 — of water in pipes, 119
 — summation curves, 54
 Forebay, 93, 148, 158
 — capacity, 60
 Forebay-regulation basin, 31, 41, 118
 Forests, 18, 21, 73, 201
 Foundations, 38, 46, 49
 Ft.³/sec., 8
 Full power resources of the stream, 36
 Functions of a reservoir, 48
 Fundamental requirements for good project,
 152
 Future development of water turbines, 3
 — hydro-electric systems, 155

GATE valves, 15, 65, 66
 Gaugings, 33
 Gauging methods, 73
 General information, 197
 Geological conditions, 31, 36, 41, 48, 94, 97
 Good project conditions, 151
 Governor-operated synchronous bye-pass, 15
 Grade or slope, 73, 99, 101, 106
 Granting of water rights, 205
 Gravity, 8
 Gravity conduits, 39, 93, 115
 Ground storage, 64, 65, 72, 73

$\sqrt{H}^3$ values, 79
 $\sqrt{H}$ h.p. constants for different condi-
 tions, 91
 "Header," 121, 128
 Head loss, 12, 41, 103, 111, 117, 130
 Head-race, 124, 148
 Head-works location, 31, 38, 148
 High-head developments, 124, 125, 127
 — impulse wheels, 14
 High over-all efficiency, 154
 — pressure turbines, 9, 124
 High-water periods, 5

High-voltage transmission, 208
 Hillside conditions, 100, 121
 — for pipe line, 39, 117
 Humidity, 27
 Hydro-electric projects, 189
 Hydraulic gradient, 123
 — losses, 100, 143-162
 — mean radius, 98, 105, 120, 146
 — properties of channels, 96-97, 100
 Hydrographs, 52
 Hydrometric survey, 32, 143
 Hydrostatic head, 11, 146

IMPERVIOUSNESS of ground, 27, 101,
 142
 Impulse type of turbine, 9, 14, 149
 Induction generator, 168, 170, 174, 178
 Inertia, 8
 Independent study, 5
 Intake loss of head, 99
 — of conduit, 115
 Intensity of rainfall, 26
 Investment costs, 200
 Investigation engineer, 23, 72, 189
 — and examinations, 32, 71, 72, 188
 — of sites, 33, 189

JETS, 9
 Joints, 41, 122, 124
 Joint efficiency, 126, 132, 141

KW. constants for different conditions, 91
 Keeping operating records, 150
 Kinetic energy, 8, 9
 kVA. capacity, 255-265

LARGE water turbines, 2, 4
 Lap joints, 41, 132, 135, 139, 141
 Leakage, 13, 60, 143, 146
 Legal aspects, 36
 Limit of depth of conduit, 99
 Lining of conduit, 100, 111
 Load factor, 43, 62
 Location of conduit, 31, 98
 — of forebay, 31
 — of pipe line, 31, 131
 — of head-works, 31
 — of power house, 31
 Longitudinal slope of channel, 98
 — section of stream, 71
 Long-period gauges, 18
 Loss of head, 41, 103, 111, 117
 Losses in riveted pipe, 132
 Low-head developments, 57, 124
 — plants, 10
 Low water, 6, 64

MAINTENANCE, 199, 223, 262
 Maintenance of conduit, 100
 Maps, 23, 194
 Masonry dams, 50

Maximum size of reaction type turbines, 14
 — stream flow, 20, 70
 — velocity, 99
 Mean hydraulic radius, 104, 109, 131, 146
 — rainfall, 18
 Measurement of catchment area, 24
 — of reservoir area, 54
 — of stream flow, 73
 — of velocity of flow, 80
 Metering stations, 32
 Most economical sections, 127
 — favourable conduit cross-section, 105
 Mountainous regions, 18, 21, 26, 27, 37, 55

NOZZLE, 9, 14

OBSTRUCTIONS and leakage, 151
 — Operating records, 150
 Ordinary mean stream flow, 35, 64
 Over-all efficiency, 5, 6, 9, 42, 147, 154, 156
 Over-development, 35, 72, 183
 — estimation, 186
 Overhead electric power transmission, 225, 265

PARALLEL operation of transmission lines, 99
 Penstocks, 93
 Percolation, 73
 Photographs, 194
 Piers, 117
 Pipe lines, 41, 93, 158
 — line installations, 119, 141
 — — location, 31
 — slope, 121
 — thickness, 122, 123
 Pitch or varnished coated cement section, 105
 Plans and maps, 194
 Plant efficiency, 13, 161
 Plants located on stream of different run-off characteristics, 4
 Plotting of typical curves for streams, 26
 Pondage, 57, 59, 149, 157
 Porosity of rock, 30
 — of soil, 64
 Power capacity, 44, 161
 — estimates, 24
 Power-house location, 31
 Power loss, 103, 165
 — producing possibilities, 52
 Practical limit of depth, 99
 Precipitation, 19, 21, 27, 45, 64, 184
 Preliminary examinations, 32, 39
 — survey, 185
 Pressure, 8
 — head, 12
 — pipe lines, 114, 118
 — regulation, 121

Primary power, 63, 153
 Porosity of water, 202
 Profile of scheme, 196

Q UANTITY of water, 80
 Quantity of water absorbed by soils, 30

$\sqrt{R}$ values, 84
 Racks or screens, 116, 117
 Rain-gauge records, 20, 21
 Rainfall, 16, 26, 201
 Rate of flow of streams, 71
 Ratio of evaporation to rainfall, 22
 — of the driest year, 19, 29
 — of the wettest year, 29
 Reaction type turbine, 10
 Reconnaissance, 37
 Recording plant output, 161
 Rectangular shaped weir, 76, 97
 Regulating basin location, 31
 Relief valves, 121
 Reporting on a water power, 184-200
 Reservoir, 48, 68, 72, 157
 — capacity, 54, 63, 143
 Rights of land, 36
 — of water, 204
 Riparian proprietor, 203
 Riveted steel pipes, 122
 Roman hydraulic works, 1
 Roughness factor, 86, 95, 106, 128
 Run-off, 16, 23
 — records, 18

$\sqrt{S}$ values, 89, 112
 Safe maximum velocities, 99
 Safe working velocity, 99
 Salt method of measuring flow, 78
 Satisfactory gauging methods, 74
 Selection of water-power sites, 31
 Screens or strainers, 115
 Secondary power, 4, 63, 153
 Secpage, 45, 54, 72, 98, 99, 100, 101, 142, 146
 Seven-day period of lowest flow, 4
 Shape of conduit, 97, 99, 101, 105-107
 — of catchment area, 17
 Short period gauges, 18
 Side slopes of conduit, 105, 117
 Single nozzle, 14
 Slight slope, 98
 Slope or grade of conduit, 73, 99, 101, 106, 117
 — measurements, 73, 86
 Sluice gates, 115
 Soil saturation, 22, 64, 65, 73
 Specific speed, 10, 12
 Speed regulation, 121
 — of water in supply pipe, 130
 Square root of the head, 13
 Stage of river or stream, 70, 95, 98
 Standard sharp crested weir, 74
 Standpipe, 123

Steam plant, 39, 92, 153
 Steep slopes, 99
 Stilling racks, 76
 Storage, 16, 47, 52, 57, 63, 69
 — capacity, 18, 48
 Stream flow, 18, 22, 52, 64, 70, 143
 — gauging, 19
 Strainers or screens in conduit, 115
 Strength of steel pipes, 123
 Summarisation of water-power report, 196
 Surges, 121
 Surge tanks, 15, 118, 158
 Survey, 32, 37, 38, 48, 55, 71, 94, 155

TAIL-RACE, 9, 10, 45, 147, 149, 159
 Tail-water, 10, 11, 45, 149
 Tail-water velocity, 149
 Temperature, 22, 26, 92, 123, 143
 Theoretical horse-power, 13
 Thickness of pipe, 117, 125, 131
 Three-halves power of the head, 13
 Thrust-bearing problem, 3, 14
 Time-flow curves, 52
 Topographic conditions, 27, 31, 41, 48, 94, 98, 110
 Transmission line design, 220, 226
 Trapezoidal methods of measuring flow, 74, 76
 — rule, 24
 — section of conduit, 98
 Turbines, 2, 3, 7

UNDERGROUND flow, 17, 64
 Units of largest size, 2
 Unlined earth sections, 99

"V" notch for measuring flow, 74, 76
 Velocity area method, 11, 73
 Velocity head, 12
 — of flow, 8, 71, 80-82, 96, 106, 121
 — in supply pipe, 121, 130
 — of tail-water, 117
 Vertical shaft wheels, 9, 14
 Volumetric or bulk method of gauging, 72

WASTE, 60
 Water-hammer, 117, 118, 121
 Water horse-power, 9, 13
 — leakage, 142
 — legislation, 201
 Water-power development, 1, 15, 30, 187, 256
 — investigations, 188
 — sites, 39
 Water rights, 36, 204
 — table, 64, 73
 Weight of rock and earth, 46, 117
 — of steel pipe, 135
 Weirs, 74
 Welded pipe, 122, 123, 132, 133, 136, 139
 Wetted perimeter, 73, 94, 105, 106, 114
 Wood-stave pipe, 135

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